AASHTO T-3 TRIAL DESIGN BRIDGE DESCRIPTION

State: *Missouri*

Trial Design Designation: MO-1

Bridge Name: *Bridge Over Rte. 60*

Superstructure Type: *Continuous steel plate girder with composite concrete deck*

Span Length(s): *Four spans (ft.) 29.5-125.3-125.3-29.5 and reinforced concrete end Slab approaches*

Substructure Type: *Three reinforced concrete columns @ Bent 2*

Foundation: *Cast-in-place reinforced concrete piles*

Abutments: *Seat type supported on a pile cap*

Seismic Design Category (SDC): *"B" however "D" assumed for trial design*

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

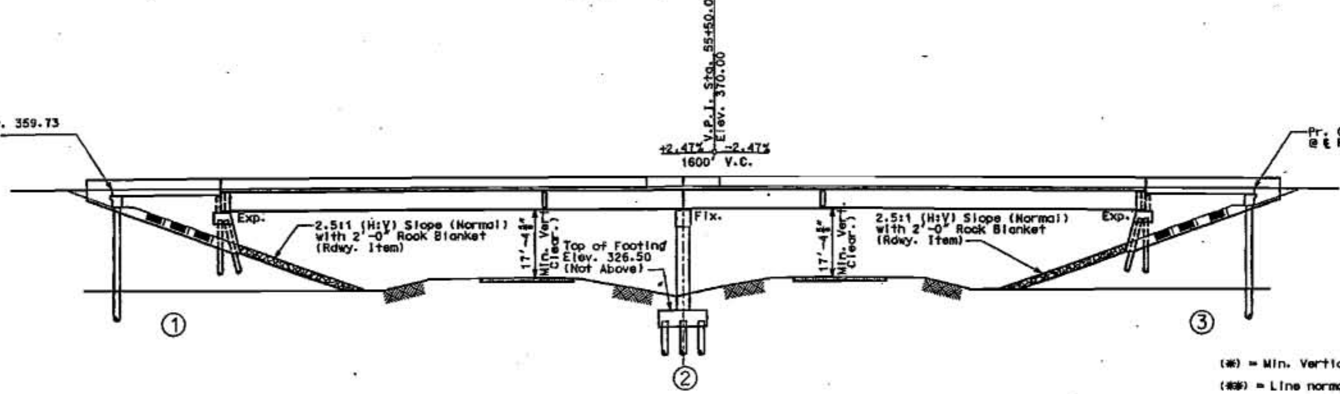
Design Spectral Acceleration at 1-second Period (S_{D1}): *0.25*

Additional Description (Optional): *Trial design has been completed and submitted and is currently being reviewed.*

State	Proj. No.	Sheet No.
MO		
SEC/SUR 27.28 TWP 25 RGE 6		

MISSOURI HIGHWAY AND TRANSPORTATION COMMISSION

(124' - 124') Continuous Composite Plate Girder Spans

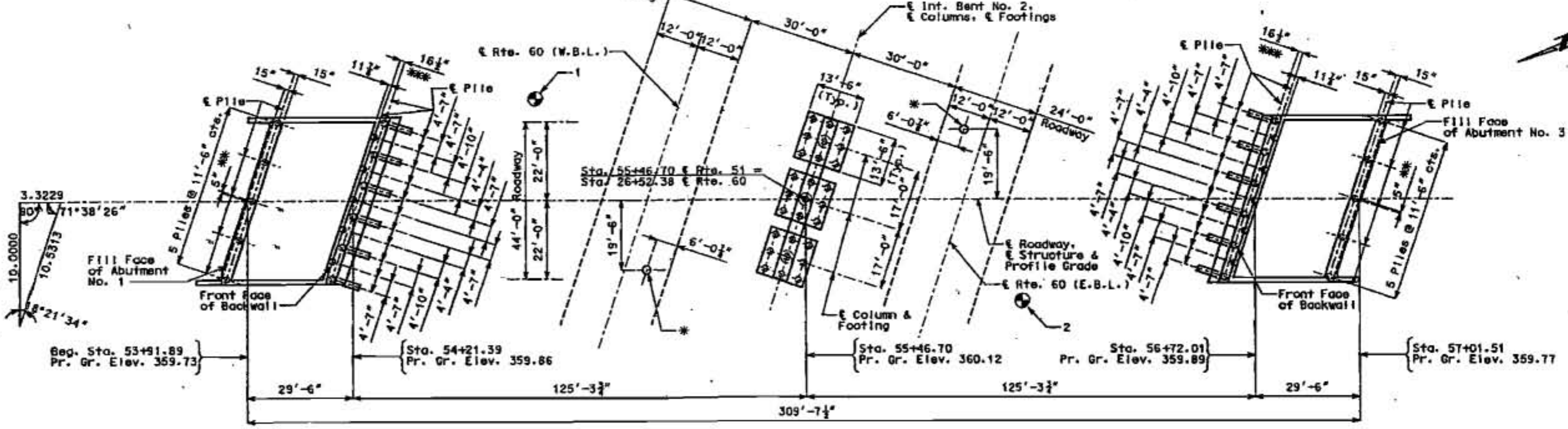


GENERAL ELEVATION

(*) = Min. Vertical Clearance

(**) = Line normal to fill face at ℄ Roadway

(***) = Dimension at bottom of bearing beam



PLAN

"⊕" Indicates location of borings.

The locations of all subsurface borings for this structure are shown on the bridge plan sheet for this structure. Boring data for the numbered locations is shown on sheet no. 3. The boring data for all locations indicated, as well as any other boring logs or other factual records of subsurface data and investigations performed by the department for the design of the project, is available from the Project Contact upon written request as outlined in the Project Special Provisions. No greater significance or weight should be given to the boring data depicted on the plan sheets than is subsurface data available from the district or elsewhere.

The Commission does not represent or warrant that any such boring data accurately depicts the conditions to be encountered in constructing this project. A contractor assumes all risks it may encounter in basing its bid prices, time or schedule of performance on the boring data depicted here or those available from the district, or on any other documentation not expressly warranted, which the contractor may obtain from the Commission.

Roadway fill shall be completed to the final roadway section and up to the elevation of the bottom of the concrete approach beam within the limits of the structure and for not less than 25 feet in back of the fill face of the end bents before any piles are driven for any bents falling within the embankment section.

Note: For General Notes, Pile Data, Estimated Quantities, Estimated Quantities for Slab on Steel and Estimated Quantites for Slab on Semi-Deep Abutments, see Sheet No. 2.

B.M. #6 Elev. 333.57 □ Top N End RCP FES 25.1' RT Sta. 54+68.2 ℄ Reloc Rte. 51

BRIDGE OVER RTE. 60

STATE ROAD

ABOUT

PROJECT NO. STA. 55+46.70

JOB NO. J0P0572E RTE. 51

STD. 609.00
STD. 611.60
STD. 702.02
STD. 706.35

Designed Jan. 2005
Detailed Dec. 2004

State	Proj. No.	Sheet No.
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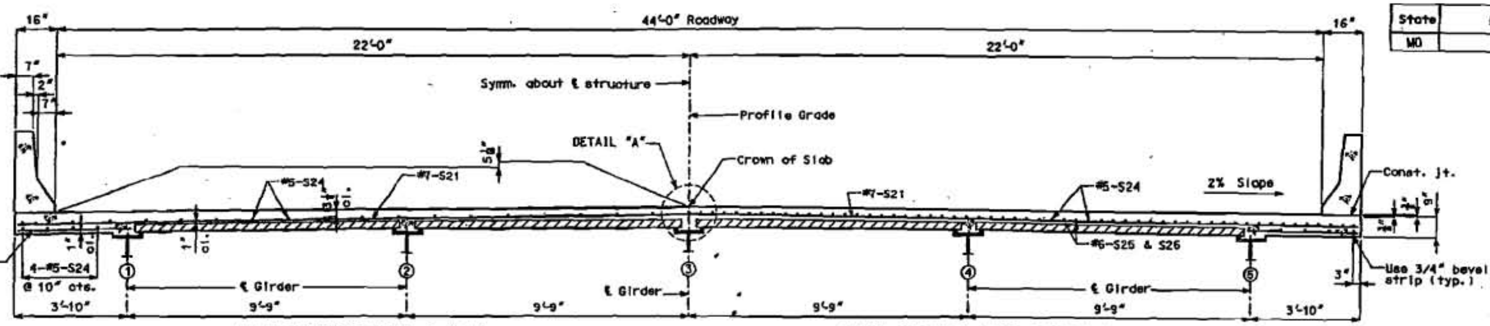
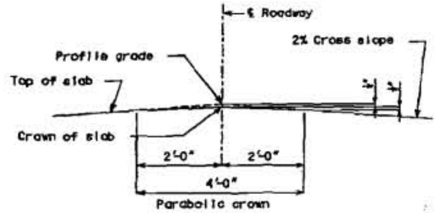
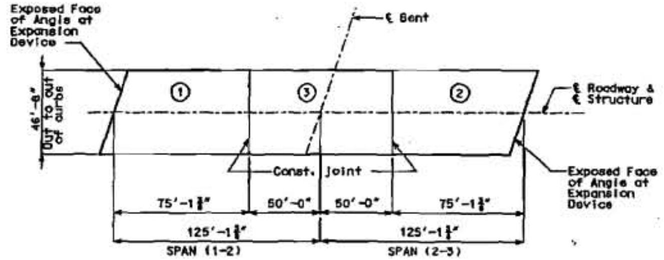


HALF SECTION NEAR ℄ SPAN   HALF SECTION NEAR INTERMEDIATE BENT

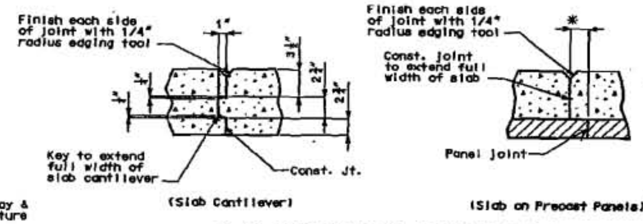
(*) Alternate bar shape available, see Safety Barrier Curb sheet.



DETAIL "A"



* Adjust the construction joint to a clearance of 6 inches minimum from the panel joint.



SLAB CONSTRUCTION JOINT DETAILS

	Sequence of Pours			Min. rate of pour cu. yds./hr.	
	Direction			With retarder	No retarder
Basic sequence	1	2	3	25	25
	Either direction				
Alternate pours to the basic skip sequence are subject to the approval of the engineer in accordance with Sec 703.					
Alternate "A" pours	1	3 + 2		50	50
	End to 3	1 to end			
Alternate "B" pours	1 + 3 + 2			30	30
	End to end				

Note: The contractor shall pour and satisfactorily finish the slab pours at the rate given. Retarder, if used, shall be an approved type and retard the set of concrete to 2.5 hours.

SLAB POURING SEQUENCE

Note:

For Plan of Slab Showing Reinforcement, see Sheet No. 28.

For details of precast prestressed panels, see Sheet No. 24.

For details and reinforcement of safety barrier curbs not shown, see Sheets No. 29, 30, and 31.

For Theoretical Bottom of Slab Elevations, see Sheet No. 18.

For Theoretical Slab Haunching Diagram and Girder Camber Diagram, see Sheet No. 18.

For details and location of Slab Drains, see Sheet No. 25.

State	Proj. No.	Sheet No.
MO		

NOTE: All reinforcing bars in the substructure beam or caps shall be spaced to clear anchor bolt wells for bearings by at least 1/2".

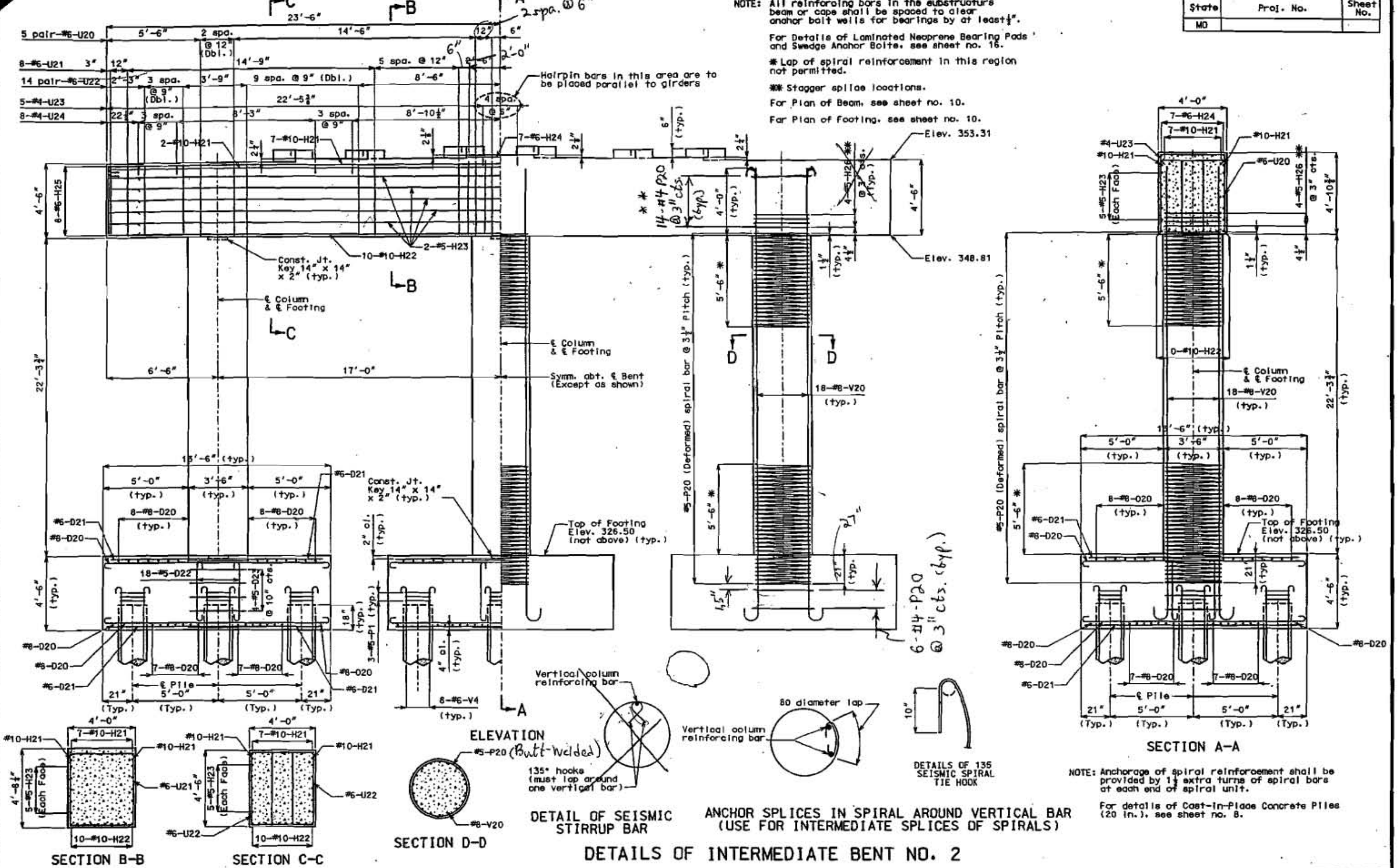
For Details of Laminated Neoprene Bearing Pads and Swedge Anchor Bolts, see sheet no. 16.

* Lap of spiral reinforcement in this region not permitted.

** Stagger splice locations.

For Plan of Beam, see sheet no. 10.

For Plan of Footing, see sheet no. 10.



NOTE: Anchorage of spiral reinforcement shall be provided by 1 1/2 extra turns of spiral bars at each end of spiral unit.

For details of Cast-In-Place Concrete Piles (20 in.), see sheet no. 8.

DETAILS OF INTERMEDIATE BENT NO. 2

Detailed
Checked

Note: This drawing is not to scale. Follow dimensions.

REV.

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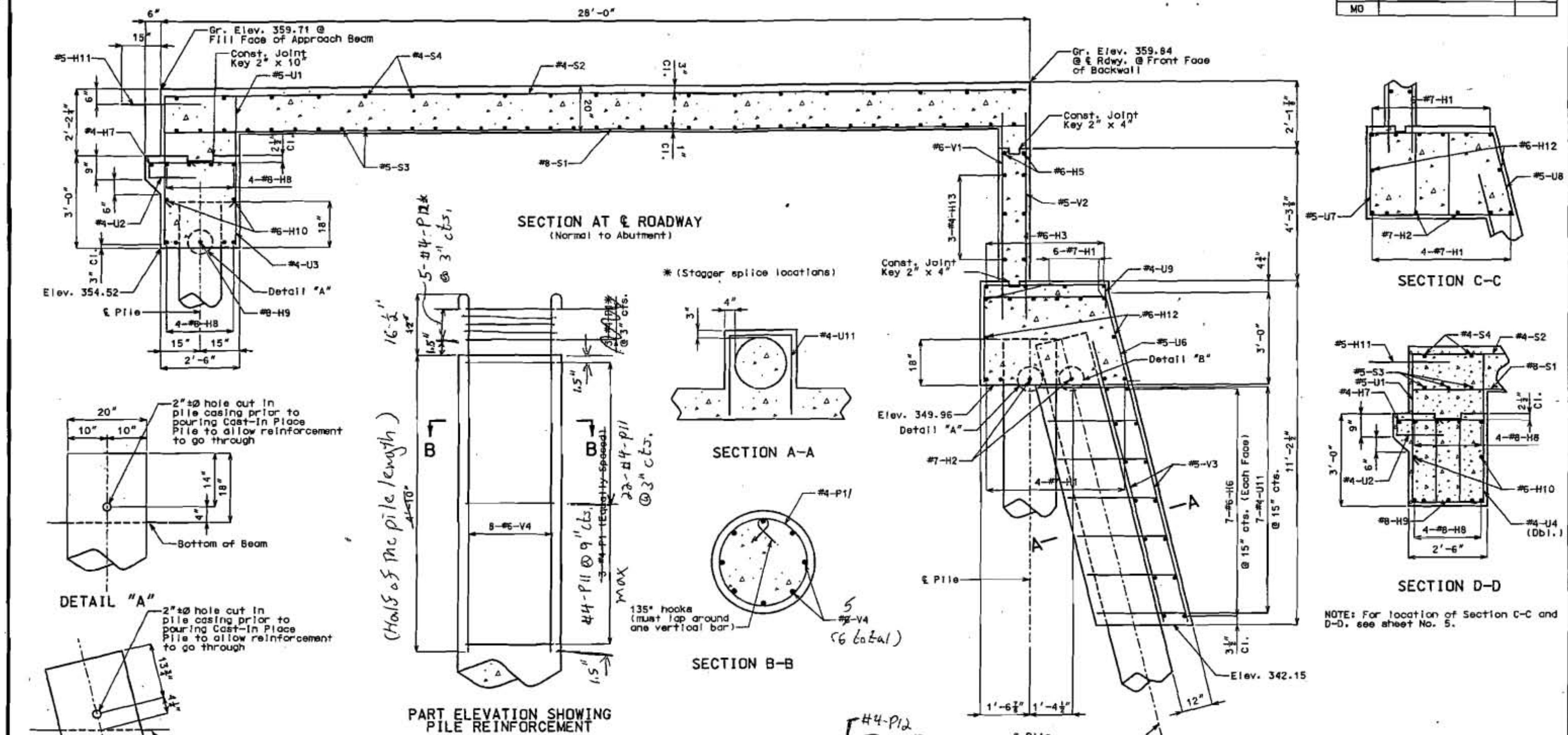
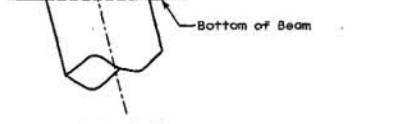
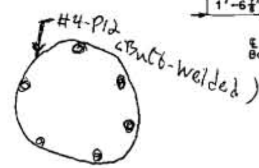
State	Proj. No.	Sheet No.
MD		







NOTE: For location of Section C-C and D-D, see sheet No. 5.

All concrete for cast-in-place piles shall be Class B-1.

Additional thickness may be required for thin shelled types to provide sufficient strength to withstand driving without injury and to resist harmful distortion or buckling due to soil pressure after being driven and the mandrel removed.

Splice details for cast-in-place concrete piles shall be in accordance with the manufacturer's recommendations.

Waterjetting will be permitted with 20" or 24" piles.

The minimum wall thickness of any spot or local area of any type shall not be more than 12.5% under the specified nominal wall thickness.

Substructure Quantity Table for Abutment 1		
Item		Quantity
Class 1 Excavation	cu. yard	50
Cast-In-Place Concrete piles (20in.)	linear foot	1,327
Pre-Bore for Piling	linear foot	314
Pile Point Reinforcement	each	16
Class B Concrete (Substructure)	cu. yard	100
Reinforcing Steel (Bridges)	pound	3,340
Reinforcing Steel (Epoxy Coated)	pound	9,020

Note: These quantities are included in the estimated quantities table on sheet no. 2.

Assume 14" pile is used!

DETAILS OF ABUTMENT NO. 1

Detailed
Checked

Note: This drawing is not to scale. Follow dimensions.

REV.

Push-over Analysis of 3-column bent in Transverse Direction

Assumption: pinned at bottom of column - fixed at bottom of beam
Equivalent cantilever analysis used ends of cantilever described above
Category D seismic zone
Liquefaction is not a concern
Self-weight of bent is ignored

Bridge was designed per LFD specifications. Pushover analysis is used to review bridge per Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (LRFD Seismic)

$kcf := \frac{kip}{ft^3}$ $\quad k := 1000lbf$

Definition of Variables (in order of appearance). Note: throughout this spreadsheet the subscripts L and R appear representing the left and right column in the bent. The center column does not use an identifier.

B = beam width
Bm_Ht = design beam height
D = column diameter
Col_Ht = clear column height (top of footing to bottom of beam)
clear_cover = concrete cover to spiral reinforcement
Col_Bar = bar size for longitudinal column reinforcement
Spiral_Bar = bar size of spiral reinforcement
Bar_Num = number of longitudinal column bars
s_v = spiral reinforcing pitch
f_c= 28 day compressive strength
f_{ce}= expected concrete compressive strength
ε_{co}= unconfined concrete compressive strain at the maximum compressive stress
ε_y= unconfined concrete yield strain
ε_{sp}= ultimate unconfined compressive (spalling) strain
γ_c= unit weight of plain concrete
E_c= plain concrete modulus of elasticity
f_y= minimum yield strength of reinforcing steel
f_{ye}= expected yield strength of reinforcing steel
f_{ue}= expected tensile strength of reinforcing steel
ε_{sh}= reinforcement strain at the onset of hardening
ε_{su}= ultimate strain of longitudinal column reinforcement
L_{col}= equivalent column length for cantilever analysis
d_b= bar diameter of longitudinal column reinforcement
L_p= plastic hinge length
DL_Super_{total}= total dead load reaction applied to bent from superstructure
N_{col} = number of columns at bent
$P_{initial}$= initial dead load reaction per column (evenly distributed)
M_p= effective yield moment of column
ϕ_y= effective yield curvature of column
ϕ_u= ultimate curvature of column
K_{eff} = effective stiffness of individual column normal to the bent.

S_{col}= column spacing parallel to bent
P_{adj}= tensile or compressive adjustment to column axial load from overturning
ω = frequency of oscillations
T_{bent}= Period of bent in transverse direction
S_a, C_s= Spectral Acceleration in magnitude and as a coefficient of g, respectively
Δ_D= displacement demand for bent in transverse direction
V_p= Shear force when column cross section becomes plastic
Δ_y= displacement between top and bottom of column at first yield
μ_D= ductility demand
θ_p= plastic rotational capacity of hinge
Δ_p= plastic displacement of column
Δ_u= ultimate displacement capacity of column
Δ_C= ultimate displacement capacity of bent
Class = Site class of soil per LRFD Seismic 3.4.2.1
A_g = gross area of concrete column
A_e = effective area of column assuming spalling of cover occurs
d_{sp} , A_{sp} = diameter and area of spiral reinforcement bar, respectively
ρ_s = volumetric ratio of spiral reinforcement for a circular column
v_c = plain concrete shear stress capacity inside plastic hinge zone
V_c = concrete column shear capacity
D` = core diameter of column
V_s = shear reinforcement capacity
V_n = total shear capacity of reinforced concrete column
V_D = shear demand associated with factored plastic moment
I_{cr} = cracked moment of inertia used in CAPP analysis
Cant = beam cap cantilever measured from centerline of exterior column
x_{ni}, y_{ni} = x and y coordinates of i^{th} joint in CAPP model
w_{bm} = dead load of superstructure distributed over beam cap length
$P_{dl}\Delta_r$ = Moment due to maximum DL column reaction and demand displacement

B := 4ft (Beam Depth)

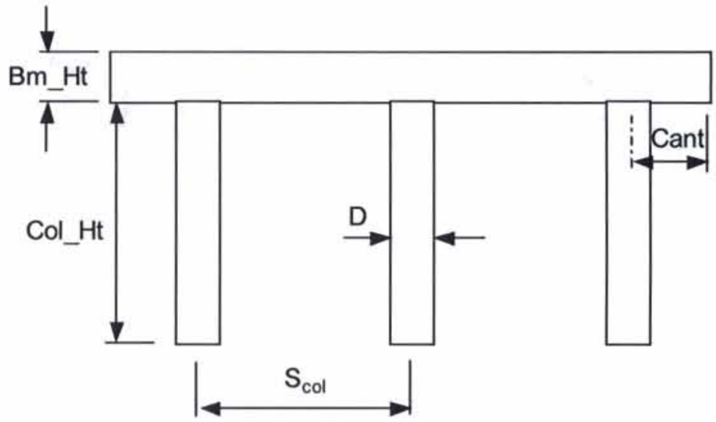
Bm_Ht := 4.5ft

Cant := 6.5ft

S_{col} := 17ft

D := 3.5ft

Col_Ht := 22.3125ft



Reinforcement:

clear_cover := 1.5in (column and beam)

Col_Bar := 8 <====Please enter numerical bar size===> Spiral_Bar := 5

$Bar\_Num := 18$ <== for information only $s_v := 3.5in$ (Spiral spacing)

I. Calculate input required for Mc1 Xtract run (Material properties and axial load)

Calculate Unconfined Concrete Properties required for XTRACT input. (LRFD Seismic 8.4.4)

$f_c := 3ksi$ $\quad f1_{ce} := 1.3 \cdot f_c$ $\quad f2_{ce} := 5ksi$

$f_{ce} := \max(f1_{ce}, f2_{ce})$ $\quad f_{ce} = 5\,ksi$

$\varepsilon_{co} := 0.002$ $\quad \varepsilon_y := 0.7 \cdot \varepsilon_{co}$ $\quad \varepsilon_y = 0.0014$

$\varepsilon_{sp} := 0.005$

$\gamma_c := 0.145 \cdot kcf$ $\quad E_c := 33000 \cdot \gamma_c^{1.5} \cdot \sqrt{f_{ce} \cdot ksi} \cdot kcf^{-1.5}$ $\quad E_c = 4.074 \times 10^3\,ksi$

Note: Set failure strain at 1.0 so that concrete spalling does not control the termination of analysis. Set Tension and Post Crushing strengths to 0.

Confined Concrete properties are calculated by Xtract using Mander's model. f_{ce} and E_c are the same as for unconfined concrete.

Longitudinal Column Reinforcing Steel Properties required for XTRACT input. (LRFD Seismic 8.4.2)

$f_y := 60ksi$ $\quad f_{ye} := 1.1f_y$ $\quad f_{ye} = 66\,ksi$

$f_{ue} := 1.4f_{ye}$ $\quad f_{ue} = 92.4\,ksi$

$$\varepsilon_{sh} := \begin{array}{|l} 0.0150 \text{ if } Col\_Bar = 8 \\ 0.0125 \text{ if } Col\_Bar = 9 \\ 0.0115 \text{ if } Col\_Bar = 10 \lor Col\_Bar = 11 \\ 0.0075 \text{ if } Col\_Bar = 14 \\ 0.0050 \text{ if } Col\_Bar = 18 \end{array}$$

$\varepsilon_{sh} = 0.015$

$\varepsilon_{su} := 0.06$

Plastic Hinge Parameters (LRFD Seismic 4.11.6)

$L_{col} := Col\_Ht$ *Equivalent cantilever height* $\quad L_{col} = 267.75\,in$

$$d_b := \begin{array}{|l} 1in \text{ if } Col\_Bar = 8 \\ 1.128in \text{ if } Col\_Bar = 9 \\ 1.27in \text{ if } Col\_Bar = 10 \\ 1.41in \text{ otherwise} \end{array}$$

$f_{ye} = 66\,ksi$

$d_b = 1\,in$

$$L_{p1} := 0.08 \cdot L_{col} + 0.15 \cdot f_{ye} \cdot \frac{d_b}{ksi} \qquad L_{p2} := 0.3 \cdot f_{ye} \cdot \frac{d_b}{ksi}$$

$$L_p := \max(L_{p1}, L_{p2}) \qquad L_p = 31.32\,in$$

Initial column reaction from dead load (same as final for center column in 3-column bent)

$$DL\_Super_{total} := 1440 \cdot k \qquad DL\_Sub_{total} := 0k \qquad DL_{total} := DL\_Super_{total} + DL\_Sub_{total}$$

$$N_{col} := 3 \qquad P_{initial} := \frac{DL_{total}}{N_{col}} \qquad P_{initial} = 480\,k$$

II. *Run Xtract Mc1 to retrieve effective yield moment and yield curvature*

$$M_p := 1830k \cdot ft \qquad \phi_y := 0.001368 \cdot \frac{1}{ft}$$

$\phi_u := 0.02435 \cdot \frac{1}{ft}$ <=== value will be used in step 7

III. *Use M_p to find adjusted axial forces for overturning.*

$$S_{col} = 17\,ft$$

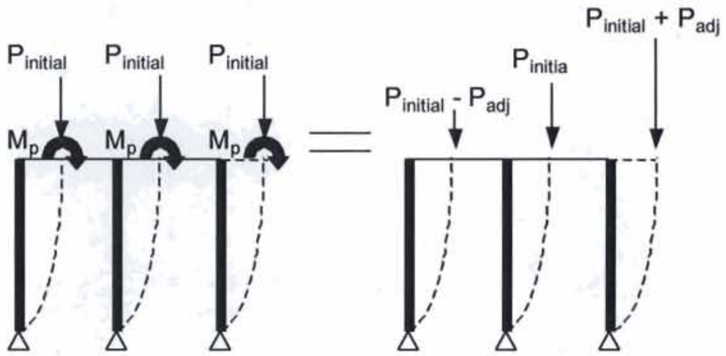


Figure 1 - Graphical derivation of overturning loads

$$P_{adj} := \frac{N_{col} \cdot M_p}{(N_{col} - 1) \cdot S_{col}}$$

$$P_{adj} = 161\,k$$

$P_L := P_{initial} - P_{adj}$ $\qquad P_L = 319\,k$ (use in Mc2)

$P_R := P_{initial} + P_{adj}$ $\qquad P_R = 641\,k$ (use in Mc3)

Check max axial load allowed for ductile design (LRFD Seismic 8.7.2)

$$A_g := \pi \cdot \frac{D^2}{4} \qquad A_g = 1385\,in^2$$

$$P_{max} := 0.2 \cdot f_{ce} \cdot A_g \qquad P_{max} = 1.385 \times 10^3\,k \qquad P_{check} := \begin{cases} \text{"O.K."} & \text{if } P_R < P_{max} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$$\boxed{P_{check} = \text{"O.K."}}$$

IV. *Run Xtract Mc2 and Mc3 files for left and right columns respectively*

$M_{pL} := 1649 k \cdot ft$ $\phi_{yL} := 0.001338 \cdot \frac{1}{ft}$ $M_{pR} := 1989 k \cdot ft$ $\phi_{yR} := 0.001388 \cdot \frac{1}{ft}$

$\phi_{uL} := 0.02383 \cdot \frac{1}{ft}$ <=== value will be used in step 7 ==> $\phi_{uR} := 0.02340 \cdot \frac{1}{ft}$

V. *Calculate displacement demand, Δ_D, of Bent.*

Assume the substructure dead load is ignored in analysis.

$DL_{total} = 1.44 \times 10^3 k$

Left Column	Center Column	Right Column	
$EI_{effL} := \frac{M_{pL}}{\phi_{yL}}$	$EI_{eff} := \frac{M_p}{\phi_y}$	$EI_{effR} := \frac{M_{pR}}{\phi_{yR}}$	LRFD Seismic 5.6.2
$EI_{effL} = 1.232 \times 10^6 k \cdot ft^2$	$EI_{eff} = 1.338 \times 10^6 k \cdot ft^2$	$EI_{effR} = 1.433 \times 10^6 k \cdot ft^2$	
$K_{effL} := 3 \cdot \frac{EI_{effL}}{L_{col}^3}$	$K_{eff} := 3 \frac{EI_{eff}}{L_{col}^3}$	$K_{effR} := 3 \cdot \frac{EI_{effR}}{L_{col}^3}$	
$K_{effL} = 333 \frac{k}{ft}$	$K_{eff} = 361 \frac{k}{ft}$	$K_{effR} = 387 \frac{k}{ft}$	

$K_{total} := K_{effL} + K_{eff} + K_{effR}$ $K_{total} = 1081 \frac{k}{ft}$

$\omega := \left(32.2 \frac{ft}{s^2} \cdot \frac{K_{total}}{DL_{total}}\right)^{0.5}$ $\omega = 4.917 \frac{1}{s}$ $T_{bent} := 2 \cdot \frac{\pi}{\omega}$ $T_{bent} = 1.278 s$

S_u values range from 1 to 1.5 ksf (15.7' - 75.9') & N_{60} values range from 9 to 41.2. These values most closely fit site class D. (LRFD Seismic 3.4.2.1)

$S_a := 6.26 \frac{ft}{s^2}$ <=== see "Seismic Design Response Spectrum" spreadsheet

$C_s := \frac{S_a}{32.2 \frac{ft}{s^2}}$ $C_s = 0.194$

$\Delta_D := DL_{total} \cdot \frac{C_s}{K_{total}}$ $\Delta_D = 3.1 in$

VI. *Check ductility demand, μ_D, of Bent. Total displacement demand vs. yield displacements.*

Calculate deflection at yield for each column. Assume hinge is located at top or bottom of column.

Left Column

$$V_{pL} := \frac{M_{pL}}{L_{col}}$$

$$V_{pL} = 73.9\,k$$

$$\Delta_{yL} := \frac{V_{pL}}{K_{effL}}$$

$$\Delta_{yL} = 2.7\,in$$

$$\mu_{DL} := \frac{\Delta_D}{\Delta_{yL}}$$

$$\mu_{DL} = 1.2$$

Center Column

$$V_p := \frac{M_p}{L_{col}}$$

$$V_p = 82\,k$$

$$\Delta_y := \frac{V_p}{K_{eff}}$$

$$\Delta_y = 2.7\,in$$

$$\mu_D := \frac{\Delta_D}{\Delta_y}$$

$$\mu_D = 1.1$$

Right Column

$$V_{pR} := \frac{M_{pR}}{L_{col}}$$

$$V_{pR} = 89.1\,k$$

$$\Delta_{yR} := \frac{V_{pR}}{K_{effR}}$$

$$\Delta_{yR} = 2.8\,in$$

$$\mu_{DR} := \frac{\Delta_D}{\Delta_{yR}}$$

$$\mu_{DR} = 1.1$$

$\mu_{max} := 8$ <=== upper limit for multi-column bents (LRFD seismic 4.9)

$$\mu_{Dcheck} := \begin{cases} \text{"O.K."} & \text{if } \mu_{DL} < \mu_{max} \wedge \mu_D < \mu_{max} \wedge \mu_{DR} < \mu_{max} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$$\boxed{\mu_{Dcheck} = \text{"O.K."}}$$

VII. *Check displacement capacity, Δ_C, of Bent. (LRFD Seismic 4.8, 4.7.1)*

Left Column

$$\theta_{pL} := (\phi_{uL} - \phi_{yL}) \cdot L_p$$

$$\theta_{pL} = 0.059$$

$$\Delta_{pL} := \theta_{pL} \cdot \left(L_{col} - \frac{L_p}{2}\right)$$

$$\Delta_{pL} = 14.799\,in$$

$$\Delta_{uL} := \Delta_{pL} + \Delta_{yL}$$

$$\Delta_{uL} = 17.463\,in$$

Center Column

$$\theta_p := (\phi_u - \phi_y) \cdot L_p$$

$$\theta_p = 0.06$$

$$\Delta_p := \theta_p \cdot \left(L_{col} - \frac{L_p}{2}\right)$$

$$\Delta_p = 15.121\,in$$

$$\Delta_u := \Delta_p + \Delta_y$$

$$\Delta_u = 17.845\,in$$

Right Column

$$\theta_{pR} := (\phi_{uR} - \phi_{yR}) \cdot L_p$$

$$\theta_{pR} = 0.057$$

$$\Delta_{pR} := \theta_{pR} \cdot \left(L_{col} - \frac{L_p}{2}\right)$$

$$\Delta_{pR} = 14.483\,in$$

$$\Delta_{uR} := \Delta_{pR} + \Delta_{yR}$$

$$\Delta_{uR} = 17.247\,in$$

$\Delta_C := \min(\Delta_{uL}, \Delta_u, \Delta_{uR})$ $\quad\quad \Delta_C = 17.2\,in$

$$\Delta_{Ccheck} := \begin{vmatrix} \text{"O.K."} & \text{if } \Delta_D < \Delta_C \\ \text{"N.G."} & \text{otherwise} \end{vmatrix}$$

$\boxed{\Delta_{Ccheck} = \text{"O.K."}}$

Ductility Capacity check (LRFD Seismic 4.7.1a)

Left Column	**Center Column**	**Right Column**
$\mu_{cL} := \dfrac{\Delta_{uL}}{\Delta_{yL}}$	$\mu_c := \dfrac{\Delta_u}{\Delta_y}$	$\mu_{cR} := \dfrac{\Delta_{uR}}{\Delta_{yR}}$
$\mu_{cL} = 6.554$	$\mu_c = 6.551$	$\mu_{cR} = 6.24$

$\mu_{cmax} := \max(\mu_{cL}, \mu_c, \mu_{cR})$

$\mu_{cmin} := \min(\mu_{cL}, \mu_c, \mu_{cR})$

$\mu_{min} := 4$ $\quad\quad \mu_{max} = 8$

$$\mu_{Ccheck} := \begin{vmatrix} \text{"O.K."} & \text{if } \mu_{cmax} < \mu_{max} \wedge \mu_{cmin} > \mu_{min} \\ \text{"N.G."} & \text{otherwise} \end{vmatrix}$$

$\boxed{\mu_{Ccheck} = \text{"O.K."}}$ <=== If O.K., then considered a full ductility structure.

VIII. *Check shear demand, V_D, to capacity, V_n, ratio. (LRFD Seismic 4.11.2, 8.6.2 & 3)*

Calculate shear capacity of each column, V_n:

$Class := \text{"D"}$ $\quad\quad D = 3.5\,ft$ $\quad\quad A_g = 1385\,in^2$ $\quad\quad A_e := 0.8A_g$

$Spiral\_Bar = 5$ $\quad\quad s_v = 3.5\,in$

$$d_{sp} := \begin{vmatrix} 0.5in & \text{if } Spiral\_Bar = 4 \\ 0.625in & \text{if } Spiral\_Bar = 5 \end{vmatrix} \quad\quad A_{sp} := \pi\cdot\left(\frac{d_{sp}}{2}\right)^2 \quad\quad A_{sp} = 0.307\,in^2$$

$D' := D - 2\cdot clear\_cover$ $\quad\quad D' = 39\,in$

$\rho_s := 4\cdot\dfrac{A_{sp}}{D'\cdot s_v}$ $\quad\quad \rho_s = 8.99\times10^{-3}$

$$\alpha_L := \begin{cases} 15\rho_s \cdot f_y & \text{if Class = "B"} \\ 10 \cdot \rho_s \cdot f_y & \text{if Class = "C"} \\ \dfrac{30 \cdot \rho_s \cdot f_y}{\mu_{DL}} & \text{if Class = "D"} \end{cases}$$

$$\alpha := \begin{cases} 15\rho_s \cdot f_y & \text{if Class = "B"} \\ 10 \cdot \rho_s \cdot f_y & \text{if Class = "C"} \\ \dfrac{30 \cdot \rho_s \cdot f_y}{\mu_D} & \text{if Class = "D"} \end{cases}$$

$$\alpha_R := \begin{cases} 15\rho_s \cdot f_y & \text{if Class = "B"} \\ 10 \cdot \rho_s \cdot f_y & \text{if Class = "C"} \\ \dfrac{30 \cdot \rho_s \cdot f_y}{\mu_{DR}} & \text{if Class = "D"} \end{cases}$$

Left Column

$\alpha_L = 13.876\,\text{ksi}$

$$v_{cL1} := \frac{1}{1000}\alpha_L \cdot \left(1 + \frac{P_L}{2 \cdot \text{ksi}\, A_g}\right) \cdot \sqrt{f_{ce} \cdot 1000 \cdot \text{ksi}}^{\,-0.5} \qquad v_{cL1} = 1.094\,\text{ksi}$$

$$v_{cL2} := 0.0035 \cdot \sqrt{f_{ce} \cdot 1000 \cdot \text{ksi}} \qquad v_{cL2} = 0.247\,\text{ksi}$$

$$v_{cL} := \begin{cases} 0 & \text{if } P_L < 0 \\ v_{cL1} & \text{if } P_L \geq 0 \wedge v_{cL1} \leq v_{cL2} \\ v_{cL2} & \text{otherwise} \end{cases} \qquad v_{cL} = 0.247\,\text{ksi}$$

$$V_{cL} := v_{cL} \cdot A_e \qquad V_{cL} = 274.3\,\text{k}$$

Center Column

$\alpha = 14.188\,\text{ksi}$

$$v_{c1} := \frac{1}{1000}\alpha \cdot \left(1 + \frac{P_{initial}}{2 \cdot \text{ksi}\, A_g}\right) \cdot \sqrt{f_{ce} \cdot 1000 \cdot \text{ksi}}^{\,-0.5} \qquad v_{c1} = 1.177\,\text{ksi}$$

$$v_{c2} := 0.0035 \cdot \sqrt{f_{ce} \cdot 1000 \cdot \text{ksi}} \qquad v_{c2} = 0.247\,\text{ksi}$$

$$v_c := \begin{cases} 0 & \text{if } P_{initial} < 0 \\ v_{c1} & \text{if } P_{initial} \geq 0 \wedge v_{c1} \leq v_{c2} \\ v_{c2} & \text{otherwise} \end{cases} \qquad v_c = 0.247\,\text{ksi}$$

$$V_c := v_c \cdot A_e \qquad V_c = 274.3\,\text{k}$$

Right Column

$\alpha_R = 14.395\,\text{ksi}$

$$v_{cR1} := \frac{1}{1000}\alpha_R \cdot \left(1 + \frac{P_R}{2 \cdot \text{ksi}\, A_g}\right) \cdot \sqrt{f_{ce} \cdot 1000 \cdot \text{ksi}}^{-0.5}$$

$v_{cR1} = 1.254\,\text{ksi}$

$v_{cR2} := 0.0035 \cdot \sqrt{f_{ce} \cdot 1000 \cdot \text{ksi}}$

$v_{cR2} = 0.247\,\text{ksi}$

$$v_{cR} := \begin{vmatrix} 0 \ \text{ if } P_R < 0 \\ v_{cR1} \ \text{ if } P_R \geq 0 \wedge v_{cR1} \leq v_{cR2} \\ v_{cR2} \ \text{ otherwise} \end{vmatrix}$$

$v_{cR} = 0.247\,\text{ksi}$

$V_{cR} := v_c \cdot A_e$ $\qquad V_{cR} = 274.3\,\text{k}$

$$V_s := \min\left(8 \cdot \sqrt{f_{ce} \cdot 1000 \cdot \frac{\text{psi}^2}{\text{ksi}}} \cdot A_e, \pi \cdot A_{sp} \cdot f_y \cdot \frac{D'}{2 \cdot s_v}\right) \qquad V_s = 322.194\,\text{kip}$$

Left Column	**Center Column**	**Right Column**
$V_{nL} := V_{cL} + V_s$	$V_n := V_c + V_s$	$V_{nR} := V_{cR} + V_s$
$V_{nL} = 596.5\,\text{k}$	$V_n = 596.5\,\text{k}$	$V_{nR} = 596.5\,\text{k}$

$\phi := 0.85$

Calculate shear demand, V_D, for each column:

$V_{DL} := 1.2 \cdot \frac{M_{pL}}{L_{col}}$	$V_D := 1.2 \cdot \frac{M_p}{L_{col}}$	$V_{DR} := 1.2 \cdot \frac{M_{pR}}{L_{col}}$
$V_{DL} = 88.7\,\text{k}$	$V_D = 98.4\,\text{k}$	$V_{DR} = 107\,\text{k}$

Check $V_D/\phi V_n < 1$ to ensure that flexure governs the frame displacement capacity. If shear governs then the bent design is no good.

$$V_{Lratio} := \frac{V_{DL}}{\phi \cdot V_{nL}} \qquad V_{ratio} := \frac{V_D}{\phi \cdot V_n} \qquad V_{Rratio} := \frac{V_{DR}}{\phi \cdot V_{nR}}$$

$$V_{Lratio} = 0.175 \qquad V_{ratio} = 0.194 \qquad V_{Rratio} = 0.211$$

$$V_{check} := \begin{cases} \text{"O.K." if } V_{Lratio} < 1 \wedge V_{ratio} < 1 \wedge V_{Rratio} < 1 \\ \text{"N.G." otherwise} \end{cases}$$

$$\boxed{V_{check} = \text{"O.K."}}$$

IX. *Xtract Input for P-M interaction. (LRFD 5.10.11.4.1b)*

Create P-M interaction load name "Lc1". Set to Half-diagram and accept limiting strains. Then click on the Code Reduction button and enter the following:

For axial loads under	0	$A_g f'_{ce}$
Moment Reduction Factor	0.9	
Axial Reduction Factor	0.9	
For axial loads over	0.2	$A_g f'_{ce}$
Moment Reduction Factor	0.5	
Axial Reduction Factor	0.5	
Set Max Axial Capacity to	0.56	$A_g f'_{ce}$

P-M curve output is used in CAPP analysis for all columns.
Code Reduced curve may be used for elastic design.

X. *Additional Input for CAPP analysis.*

Column cracked moment of inertias

$$I_{crL} := \frac{EI_{effL}}{E_c} \qquad \boxed{I_{crL} = 43559\ in^4}$$

$$I_{cr} := \frac{EI_{eff}}{E_c} \qquad \boxed{I_{cr} = 47280\ in^4}$$

$$I_{crR} := \frac{EI_{effR}}{E_c} \qquad \boxed{I_{crR} = 50647\ in^4}$$

Beam cracked moment of inertia, I_x

Definitions for this section:

Beam_Stirrup = Stirrup size used in beam
Beam_Bar = Size of longitudinal bars used for flexural reinforcement
Num_Bar = number of bars used in either the top or bottom of the beam (assumed symmetric)
d_{st} = diameter of stirrup
d_{bm} = diameter of flexural reinforcement
d_{eff} = effective depth of flexural reinforcement
n = modular ratio of expected concrete strength
A_s = area of steel in either the top or bottom of the beam
x_1, x_2 = two possible solutions to the quadratic equation for x.
x = distance from the neutral axis to the extreme compression face
I_x = moment of inertia of cracked beam section

$\text{Beam\_Stirrup} := 6$

$\text{Beam\_Bar} := 10 \qquad \text{Num\_bar} := 10$

$$d_{st} := \frac{\text{Beam\_Stirrup} \cdot \text{in}}{8} \qquad d_{st} = 0.75\,\text{in}$$

$$d_{bm} := \begin{array}{|l} 1\text{in} \ \text{ if Beam\_Bar} = 8 \\ 1.128\text{in} \ \text{ if Beam\_Bar} = 9 \\ 1.27\text{in} \ \text{ if Beam\_Bar} = 10 \\ 1.41\text{in} \ \text{ otherwise} \end{array} \qquad d_{bm} = 1.27\,\text{in}$$

$$d_{eff} := \text{Bm\_Ht} - \text{clear\_cover} - d_{st} - \frac{d_{bm}}{2} \qquad d_{eff} = 51.115\,\text{in}$$

$$n := \frac{29000 \cdot \text{ksi}}{E_c} \qquad n = 7.118$$

$$A_s := \text{Num\_bar} \cdot \pi \cdot \frac{d_{bm}^{\,2}}{4} \qquad A_s = 12.67\,\text{in}^2$$

$$x_1 := -n \cdot \left(\frac{A_s}{B}\right) \cdot \left(1 + \sqrt{1 + 2 \cdot B \cdot \frac{d_{eff}}{n \cdot A_s}}\right)$$

$$x_2 := -n \cdot \left(\frac{A_s}{B}\right) \cdot \left(1 - \sqrt{1 + 2 \cdot B \cdot \frac{d_{eff}}{n \cdot A_s}}\right)$$

$$x := \begin{array}{|l} x_1 \ \text{ if } x_1 > 0 \\ x_2 \ \text{ otherwise} \end{array} \qquad x = 12.11\,\text{in}$$

$$I_x := B \cdot \left(\frac{x^3}{3}\right) + n \cdot A_s \cdot \left(d_{eff} - x\right)^2 \qquad \boxed{I_x = 165593\,\text{in}^4}$$

Node Point locations for symmetric 3-column bent

$x_{n1} := -S_{col} \qquad y_{n1} := 0$

$x_{n2} := -S_{col} \qquad y_{n2} := \dfrac{L_p}{2}$

$x_{n3} := -S_{col} \qquad y_{n3} := Col\_Ht - \dfrac{L_p}{2}$

$x_{n4} := -S_{col} \qquad y_{n4} := Col\_Ht$

$x_{n5} := -S_{col} \qquad y_{n5} := Col\_Ht + \dfrac{Bm\_Ht}{2}$

$x_{n6} := 0 \qquad y_{n6} := 0$

$x_{n7} := 0 \qquad y_{n7} := \dfrac{L_p}{2}$

$x_{n8} := 0 \qquad y_{n8} := Col\_Ht - \dfrac{L_p}{2}$

$x_{n9} := 0 \qquad y_{n9} := Col\_Ht$

$x_{n10} := 0 \qquad y_{n10} := Col\_Ht + \dfrac{Bm\_Ht}{2}$

$x_{n11} := S_{col} \qquad y_{n11} := 0$

$x_{n12} := S_{col} \qquad y_{n12} := \dfrac{L_p}{2}$

$x_{n13} := S_{col} \qquad y_{n13} := Col\_Ht - \dfrac{L_p}{2}$

$x_{n14} := S_{col} \qquad y_{n14} := Col\_Ht$

$x_{n15} := S_{col} \qquad y_{n15} := Col\_Ht + \dfrac{Bm\_Ht}{2}$

$x_{n16} := -S_{col} - Cant \qquad y_{n16} := Col\_Ht + \dfrac{Bm\_Ht}{2}$

$x_{n17} := S_{col} + Cant \qquad y_{n17} := Col\_Ht + \dfrac{Bm\_Ht}{2}$

Joint Locations

Joint 1	$x_{n1} = -17\,ft$	$y_{n1} = 0$
Joint 2	$x_{n2} = -17\,ft$	$y_{n2} = 1.305\,ft$
Joint 3	$x_{n3} = -17\,ft$	$y_{n3} = 21.008\,ft$
Joint 4	$x_{n4} = -17\,ft$	$y_{n4} = 22.313\,ft$
Joint 5	$x_{n5} = -17\,ft$	$y_{n5} = 24.563\,ft$
Joint 6	$x_{n6} = 0\,ft$	$y_{n6} = 0\,ft$
Joint 7	$x_{n7} = 0\,ft$	$y_{n7} = 1.305\,ft$
Joint 8	$x_{n8} = 0\,ft$	$y_{n8} = 21.008\,ft$
Joint 9	$x_{n9} = 0\,ft$	$y_{n9} = 22.313\,ft$
Joint 10	$x_{n10} = 0\,ft$	$y_{n10} = 24.563\,ft$
Joint 11	$x_{n11} = 17\,ft$	$y_{n11} = 0\,ft$
Joint 12	$x_{n12} = 17\,ft$	$y_{n12} = 1.305\,ft$
Joint 13	$x_{n13} = 17\,ft$	$y_{n13} = 21.008\,ft$
Joint 14	$x_{n14} = 17\,ft$	$y_{n14} = 22.313\,ft$
Joint 15	$x_{n15} = 17\,ft$	$y_{n15} = 24.563\,ft$
Joint 16	$x_{n16} = -23.5\,ft$	$y_{n16} = 24.563\,ft$
Joint 17	$x_{n17} = 23.5\,ft$	$y_{n17} = 24.563\,ft$

Find dead load to distribute to beam cap (beam cap load not included)

$$w_{bm} := \frac{DL_{total}}{2 \cdot Cant + \left(N_{col} - 1\right) \cdot S_{col}} \qquad w_{bm} = 30.638\,\frac{k}{ft}$$

Find Rotation Capacity of each column from Xtract output

$RotCap_L := 0.05872$

$RotCap := 0.05999$

$RotCap_R := 0.05746$

Check for P-Δ effects? Right column will control. (LRFD Seismic 4.11.5)

$P_{dl} := P_R \qquad \Delta_r := \Delta_D \qquad P_{dl} \cdot \Delta_r = 166.1\ k \cdot ft$

$P\Delta_{max} := 0.25 \cdot M_{pR} \qquad P\Delta_{max} = 497.3\ k \cdot ft$

$$P\Delta_{check} := \begin{cases} \text{"Not Required"} & \text{if } P_{dl} \cdot \Delta_r < P\Delta_{max} \\ \text{"Required"} & \text{otherwise} \end{cases}$$

Note: If the PΔ check is not met than a PΔ analysis is required. According to the LRFD Seismic specifications a nonlinear time history analysis is required for an accurate PΔ analysis. Since an unreasonable deflection capacity is obtained from CAPP when PΔ effects are not considerred, PΔ effects should be included in the CAPP analysis regardless of the statement below.

A P-Δ analysis is "Not Required" Check View Options in CAPP program

Detailing Calculations

Definition of Variables

$Spiral_{min}$ = minimum size of spiral reinforcement allowed in the column
s_{vmax} = maximum spacing of spiral reinforcement in the column
ρ_{smin} = minimum spiral reinforcement ratio allowed in the column
A_{bc} = Area of a longitudinal bar in the column
A_{sc} = total area of longitudinal steel in the column
A_{scmax} = maximum area of longitudinal steel allowed in the column
Min_Ratio = minimum ratio of longitudinal steel area to column section area
A_{scmin} = minimum area of longitudinal steel required in the column
PHR_1, PHR_2, PHR_3 = three values that define the limits of the Plastic Hinge Region
l_{ac_ft} = anchorage length of column longitudinal steel into footing
D_{foot} = footing depth
Foot_Cl_Cover = clear concrete cover at bottom of footing for footing flexural steel
l_{ac_bm} = anchorage length of column longitudinal steel into beam
l_{ac_min} = minimum development length of column longitudinal steel into beam
LR_Max_1, LR_Max_2 = maximum lateral reinforcement spacing values for the column
M_o = overstrength moment required to be resisted by elastic members (i.e, beam and footing)
M_{ne_bm} = nominal flexural capacity of beam
T_c = column tensile force resulting from overstrength moment
A_{jv} = effective vertical joint area (plane normal to longitudinal axis of beam)
v_{jv} = shear stress of effective vertical joint area
P_c = column axial force including effects from overturning
A_{jh} = effective horizontal joint area
f_v = vertical stress normal to horizontal plane
p_c = principal compression force on stress block (figure 3)
p_t = principal tension force on stress block (figure 3)
p_{c_max} = maximum allowable principal compression force in T-Joint
p_{t_max} = maximum allowable principal tension force in T-Joint
p_{tmin} = minimum design principal tension for lateral reinforcement design
A_{sjv_min} = minimum area of vertical stirrups in beam T-joint
N_{jv} = number of vertical stirrups in beam within a core diameter on either side of column centerline
A_{bjv} = Area of one leg of stirrup used for vertical stirrup requirement in T-joint
A_{sjv} = Area of vertical steel (stirrups) within core diameter from centerline of column
A_{s_bot} = Area of flexural reinforcement in bottom of beam
A_{s_top} = Area of flexural reinforcement in top of beam
A_{sf_min} = minimum area of longitudinal reinforcement required in sides of beam for T-Joint
N_{sf} = number of longitudinal bars present on one side of beam for T-Joint
A_{bsf} = area of a longitudinal bar in the side of the beam
A_{sf} = total area of longitudinal reinforcement on one side of beam
ρ_{s_lim} = lower limit for spiral reinforcement ratio in column

ρ_{hoop} = reinforcement ratio of hoop steel extended into beam
M_{ne_ft} = flexural capacity of footing in bent longitudinal direction
L_{foot} = length of footing (measured in bent longitudinal direction)
B_{eff_ftg} = effective footing width for T-Joint design
A_{jv_ft} = effective vertical joint area in footing (plane normal to bent centerline)
A_{jh_ft} = effective square horizontal joint area

Minimum spiral Reinforcement? (LRFD Seismic 8.4.1, 8.6.5)...Bundling bars not considered

$$Spiral_{min} := \begin{cases} 3 & \text{if } Col\_Bar < 10 \\ 5 & \text{otherwise} \end{cases} \qquad Spiral_{min} = 3$$

$$Spiral\_Size\_Check := \begin{cases} \text{"O.K."} & \text{if } Spiral\_Bar \geq Spiral_{min} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$$\boxed{Spiral\_Size\_Check = \text{"O.K."}}$$

$$s_{vmax} := \min(D, 12in) \qquad s_{vmax} = 12\,in$$

$$Spiral\_Spacing\_Check := \begin{cases} \text{"O.K."} & \text{if } s_v \leq s_{vmax} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$$\boxed{Spiral\_Spacing\_Check = \text{"O.K."}}$$

Minimum Shear Reinforcement (LRFD Seismic 8.6.6)

$\rho_s = 0.00899$ $\qquad$ $\rho_{smin} := 0.004$ <=== 0.004 for C & D, 0.002 for B

$$Reinf\_Ratio\_Check := \begin{cases} \text{"O.K."} & \text{if } \rho_s > \rho_{smin} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$$\boxed{Reinf\_Ratio\_Check = \text{"O.K."}}$$

Maximum Longitudinal Reinforcement (LRFD Seismic 8.8.1)

$$A_{bc} := \pi \cdot \frac{{d_b}^2}{4} \qquad A_{bc} = 0.785\,in^2$$

$$A_{sc} := A_{bc} \cdot Bar\_Num \qquad A_{sc} = 14.137\,in^2$$

$$A_{scmax} := 0.04 \cdot A_g \qquad A_{scmax} = 55.418\,in^2$$

$$Max\_Col\_Steel\_Check := \begin{cases} \text{"O.K."} & \text{if } A_{sc} \leq A_{scmax} \\ \text{"N.G."} & \text{otherwise} \end{cases} \qquad \boxed{Max\_Col\_Steel\_Check = \text{"O.K."}}$$

Minimum Longitudinal Reinforcement (LRFD Seismic 8.8.2)

$Min\_Ratio := 0.01$ Enter 0.007 for SDC B or C
0.01 for SDC D

$A_{scmin} := Min\_Ratio \cdot A_g$ $A_{scmin} = 13.854\ in^2$

$$Min\_Col\_Steel\_Check := \begin{cases} \text{"O.K."} & \text{if } A_{sc} \geq A_{scmin} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{Min\_Col\_Steel\_Check = \text{"O.K."}}$

Column Splicing Check (LRFD Seismic 8.8.3)...Not required for SBC B

Enter one of the following describing the location of the longitudinal steel column splice:
0 = No splice, Distance from top of column to beginning of 1st splice

$Splice\_Location := 0$

$PHR_1 := 1.5 \cdot D$

$PHR_2 := L_p$

$PHR_3 := 0.25 \cdot L_{col}$ $PHR_3 = 5.578\ ft$ <==== Distance from bottom of beam where M < 0.75*M_p

$$Sp\_Loc\_Check := \begin{cases} \text{"O.K."} & \text{if } Splice\_Location = 0 \vee Splice\_Location > \max(PHR_1, PHR_2, PHR_3) \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{Sp\_Loc\_Check = \text{"O.K."}}$

Minimum Development Length of Reinforcing Steel for SDC B, C, or D (LRFD Seismic 8.8.4)

$l_{ac\_ft} := 50in$ $D_{foot} := 54in$ $Foot\_Cl\_Cover := 4in$

$l_{ac\_bm} := 48in$

$l_{ac\_min} := 24 \cdot d_b$ $l_{ac\_min} = 24\ in$ <=== applies to beams only

$$Min\_Dev\_Check := \begin{cases} \text{"O.K."} & \text{if } l_{ac\_ft} = D_{foot} - Foot\_Cl\_Cover \wedge l_{ac\_bm} \geq l_{ac\_min} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{Min\_Dev\_Check = \text{"O.K."}}$

Maximum Column Bar Diamter for SDC B, C, or D (LRFD Seismic 8.8.6)

$$Bar\_Diameter\_Max := 25 \cdot \sqrt{f_c \cdot 1000 \cdot ksi} \cdot \frac{L_{col} - 0.5 \cdot D'}{f_{ye} \cdot 1000}$$ $Bar\_Diameter\_Max = 5\ in$

$d_b = 1\ in$

$$Bar\_Diameter\_Check := \begin{cases} \text{"O.K."} & \text{if } d_b \leq Bar\_Diameter\_Max \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{Bar\_Diameter\_Check = \text{"O.K."}}$

Lateral Reinforcement inside the Plastic Hinge Region for SDC D (LRFD Seismic 8.8.7)

V/D check satisfies this section with spirals provided in the plastic hinge region.

Minimum Stirrup outside plastic hinge region for SDC C and D (LRFD Seismic 8.8.8)

Check not required because same lateral reinforcement is used inside and outside plastic hinge region.

Maximum Spacing for Lateral reinforcement for SDC C or D (LRFD Seismic 8.8.9)

$LR\_Max_1 := 0.2 \cdot D \qquad LR\_Max_1 = 8.4\,in$

$LR\_Max_2 := 6 \cdot d_b \qquad LR\_Max_2 = 6\,in$

$s_v = 3.5\,in$

$$Spiral\_End\_Spacing\_Check := \begin{cases} \text{"O.K."} & \text{if } s_v \le LR\_Max_1 \wedge s_v \le LR\_Max_2 \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{Spiral\_End\_Spacing\_Check = \text{"O.K."}}$

Note: Lateral reinforcement shall extend to beginning of hook bend in footings.

T-Joint Connection Checks (LRFD Seismic 8.13.4)

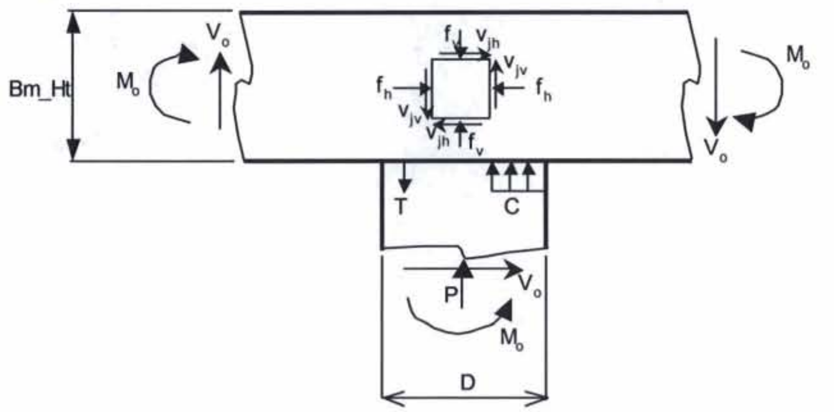


Figure 2 - Joint shear stresses in beam cap T-joint
Note: Moment (M_o) and shear (V_o) values are different in beam and column.

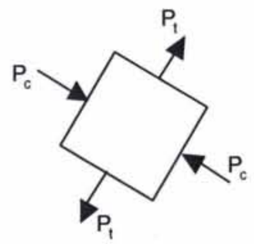


Figure 3 - Prinipal Stresses

Assumptions: 1) vertical axial stress from column is distributed to center of beam through 45 degree angles in the beam longitudinal direction.
2) vertical axial stress from column is distributed to center of beam through an effective joint width, 1.414D, in the beam transverse direction.
3) Horizontal shear stress is distributed over column diamter in beam longitudinal direction and effective joint width, 1.414D, in beam transverse direction.
4) The right column T-Joint will control.

Right Column - Beam Joint

1. First check that the beam can carry column plastic moment elastically.

$M_o := 1.2M_{pR}$ $\quad\quad M_o = 2.387 \times 10^3\,k\cdot ft$

$M_{ne\_bm} := 3031k\cdot ft$ <===negative moment flexural capacity

$$\text{Elastic\_Beam\_Check} := \begin{cases} \text{"O.K."} & \text{if } M_o < M_{ne\_bm} \\ \text{"N.G"} & \text{otherwise} \end{cases} \quad\quad \boxed{\text{Elastic\_Beam\_Check} = \text{"O.K."}}$$

2. Calculate principal forces

$$T_c := \frac{M_o}{0.7\cdot D}$$ <===Approximation of distance between tensile and compression forces in column

$T_c = 974\,k$ $\quad\quad l_{ac} := l_{ac\_bm}$

$A_{jv} := l_{ac}\cdot B$ $\quad\quad A_{jv} = 2304\,in^2$

$v_{jv} := \dfrac{T_c}{A_{jv}}$ $\quad\quad v_{jv} = 0.423\,ksi$

$P_c := P_R$ $\quad\quad P_c = 641\,k$

$A_{jh} := (D + Bm\_Ht)\cdot B$ $\quad\quad A_{jh} = 4608\,in^2$

$f_v := \dfrac{P_c}{A_{jh}}$ $\quad\quad f_v = 0.139\,ksi$

$f_h := 0$

$$p_c := \frac{f_h + f_v}{2} + \sqrt{\frac{(f_h - f_v)^2}{4} + v_{jv}^2} \quad\quad p_c = 0.498\,ksi$$

$$p_t := \frac{f_h + f_v}{2} - \sqrt{\frac{(f_h - f_v)^2}{4} + v_{jv}^2} \quad\quad p_t = -0.359\,ksi$$

Joint Proportioning Check (LRFD Seismic 8.13.2)

$p_{c\_max} := 0.25 \cdot f_{ce}$ $\qquad p_{c\_max} = 1.25\,\text{ksi}$

$p_{t\_max} := 12\sqrt{f_{ce} \cdot 1000 \cdot \frac{psi^2}{ksi}}$ $\qquad p_{t\_max} = 0.849\,\text{ksi}$

$$\text{Stress\_Check} := \begin{cases} \text{"O.K."} & \text{if } p_c \le p_{c\_max} \wedge |p_t| \le p_{t\_max} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{\text{Stress\_Check} = \text{"O.K."}}$

If the principal tension stress is less than 3.5 times the square root of the effective compressive strength of concrete the minimum joint shear requirements control the design. (LRFD Seismic 8.13.4.2) SDC C or D

$p_{tmin} := 3.5\sqrt{f_{ce} \cdot 1000 \cdot \frac{psi^2}{ksi}}$ $\qquad p_{tmin} = 0.247\,\text{ksi}$

$$\text{Use\_Min\_Shear} := \begin{cases} \text{"Yes"} & \text{if } p_t > -p_{tmin} \\ \text{"No"} & \text{otherwise} \end{cases}$$

$\text{Use\_Min\_Shear} = \text{"No"}$

$$\rho_{s\_min} := \begin{cases} \dfrac{p_{tmin}}{f_{ye}} & \text{if Use\_Min\_Shear} = \text{"Yes"} \\ 0 & \text{otherwise} \end{cases} \qquad \rho_{s\_min} = 0$$

Joint Shear Reinforcement SDC D? (LRFD Seismic 8.13.4.3)

$A_{sjv\_min} := 0.2 \cdot A_{sc}$ $\qquad A_{sjv\_min} = 2.827\,\text{in}^2$

Apply within core diameter from centerline of column

$N_{jv} := 5$ $\qquad A_{bjv} := 0.4418\text{in}^2$ $\qquad$ <=== # and size of bar on one side of column

$A_{sjv} := 2N_{jv} \cdot A_{bjv}$ $\qquad A_{sjv} = 4.418\,\text{in}^2$

$$\text{Vertical\_Steel\_Check} := \begin{cases} \text{"O.K."} & \text{if } A_{sjv} \ge A_{sjv\_min} \\ \text{"Not Required"} & \text{if Use\_Min\_Shear} = \text{"Yes"} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{\text{Vertical\_Steel\_Check} = \text{"O.K."}}$

$A_{s\_bot} := 12.656\text{in}^2$ $\qquad A_{s\_top} := 11.39\text{in}^2$

$A_{sf\_min} := 0.1 \cdot \max(A_{s\_bot}, A_{s\_top})$ $\qquad A_{sf\_min} = 1.266\,in^2$

Apply in side faces of beam with a maximum spacing of 12 in.

$N_{sf} := 5$ $\qquad A_{bsf} := 0.3068in^2$ <=== # and size of bars on one side of cap (discluding flexural steel)

$A_{sf} := N_{sf} \cdot A_{bsf}$ $\qquad A_{sf} = 1.534\,in^2$

$$\text{Side\_Steel\_Check} := \begin{array}{|l} \text{"O.K." if } A_{sf} \geq A_{sf\_min} \\ \text{"Not Required" if Use\_Min\_Shear} = \text{"Yes"} \\ \text{"N.G." otherwise} \end{array}$$

$\boxed{\text{Side\_Steel\_Check} = \text{"O.K."}}$

Assume #4 @ 3" hoops are used for transverse reinforcement in beam cap T-joint

$$\rho_{s\_lim} := \begin{array}{|l} 0.4 \cdot \dfrac{A_{sc}}{l_{ac}^2} \text{ if Use\_Min\_Shear} = \text{"No"} \\ 0 \text{ otherwise} \end{array} \qquad \rho_{s\_lim} = 2.454 \times 10^{-3}$$

$$\rho_{hoop} := 4 \cdot 0.1963in^2 \cdot \frac{D' - 0.5in}{3in \cdot D'^2} \qquad \rho_{hoop} = 6.625 \times 10^{-3}$$

$$\text{hoop\_check} := \begin{array}{|l} \text{"O.K." if } \rho_{hoop} \geq \rho_{s\_min} \wedge \rho_{hoop} \geq \rho_{s\_lim} \\ \text{"N.G." otherwise} \end{array}$$

$\boxed{\text{hoop\_check} = \text{"O.K."}}$

Right Column - Footing Joint

1. First check that the beam can carry column plastic moment elastically.

$M_o := 1.2M_{pR}$ $\qquad M_o = 2.387 \times 10^3\ k \cdot ft$

$M_{ne\_ft} := 3325k \cdot ft$ <===positive moment flexural capacity

$$Elastic\_Foot\_Check := \begin{cases} \text{"O.K."} & \text{if } M_o < M_{ne\_ft} \\ \text{"N.G"} & \text{otherwise} \end{cases}$$

$\boxed{Elastic\_Beam\_Check = \text{"O.K."}}$

2. Calculate principal forces

$T_c := \dfrac{M_o}{0.7 \cdot D}$ <===Approximation of distance between tensile and compression forces in column

$T_c = 974\ k$

$B_{eff\_ftg} := \sqrt{2} \cdot D$ $\qquad B_{eff\_ftg} = 59.397\ in$

$A_{jv\_ft} := B_{eff\_ftg} \cdot D_{foot}$ $\qquad A_{jv\_ft} = 3207\ in^2$

$v_{jv} := \dfrac{T_c}{A_{jv\_ft}}$ $\qquad v_{jv} = 0.304\ ksi$

$P_c := P_R$ $\qquad P_c = 641\ k$

$A_{jh\_ft} := (D + D_{foot})^2$ $\qquad A_{jh\_ft} = 9216\ in^2$

$f_v := \dfrac{P_c}{A_{jh\_ft}}$ $\qquad f_v = 0.07\ ksi$

$f_h := 0$

$$p_c := \frac{f_h + f_v}{2} + \sqrt{\frac{(f_h - f_v)^2}{4} + v_{jv}^2} \qquad p_c = 0.341\ ksi$$

$$p_t := \frac{f_h + f_v}{2} - \sqrt{\frac{(f_h - f_v)^2}{4} + v_{jv}^2} \qquad p_t = -0.271\ ksi$$

Check principal stresses are within limits (LRFD 6.4.5)

$p_{c\_max} := 0.25 \cdot f_{ce}$ $\qquad p_{c\_max} = 1.25\,ksi$

$p_{t\_max} := 12\sqrt{f_{ce} \cdot 1000 \cdot \frac{psi^2}{ksi}}$ $\qquad p_{t\_max} = 0.849\,ksi$

$$Stress\_Check := \begin{cases} \text{"O.K."} & \text{if } p_c \le p_{c\_max} \wedge |p_t| \le p_{t\_max} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{Stress\_Check = \text{"O.K."}}$

If the principal tension stress is less than 3.5 times the square root of the effective compressive strength of concrete the minimum joint shear requirements control the design. (LRFD Seismic 8.13.4.2) SDC C or D

$p_{tmin} := 3.5\sqrt{f_{ce} \cdot 1000 \cdot \frac{psi^2}{ksi}}$ $\qquad p_{tmin} = 0.247\,ksi$

$\rho_{s\_min} := \frac{p_{tmin}}{f_{ye}}$ $\qquad \rho_{s\_min} = 0.00375$

Assume #4 @ 3" hoops are used for transverse reinforcement in footing T-joint

$\rho_{hoop} := 4 \cdot 0.1963\,in^2 \cdot \frac{D' - 0.5in}{3in \cdot D'^2}$ $\qquad \rho_{hoop} = 6.625 \times 10^{-3}$

$$hoop\_check := \begin{cases} \text{"O.K."} & \text{if } \rho_{hoop} \ge \rho_{s\_min} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{hoop\_check = \text{"O.K."}}$

PILE DESIGN CHECKS

L_{foot} = footing dimension in bent longitudinal direction
W_{foot} = footing dimenion in bent transverse direction
N_rows_{long}, S_Pile_{tran}, N_rows_{tran}, S_Pile_{long} = See figure below
Pile_Dia = C.I.P. pile diameter measured to outside face of steel casing
A_{pile_st} = Total area of longitudinal reinforcement in C.I.P. pile (not used)
A_{shell} = Cross-sectional area of steel casing (not used by this worksheet)
I_{pgx}, I_{pgz} = Pile group moment of inertia about X and Z axis respectively
DL_Sub = Total self weight of beam cap and columns
P_{TOF_L} = Weight of super and sub at bottom of left column
P_{BOF_L}, P_{BOF_R} = Weight of super, sub, and footing at bottom of left and right footing respectively
R_{min}, R_{max} = min and max reactions on outer piles assuming a plastic moment developed about the Z or X axis.
T_{pile} = Structural tensile capacity of pile
Radius = radius of bottom of shear cone failure surface
SC_V_c = Shear cone capacity
T_{shear} = Allowable tension in pile for footing shear (F.S. = 3)
$T_{friction}$ = geotechnical friction capacity of pile in tension
T_{all} = allowable tension force in pile for pushover analysis
A_{g_pile} = Gross area of C.I.P. pile
$C_{friction}$ = bearing capacity of pile with 1.5 allowable overstress for seismic event
Vpile = Shear force per pile assuming even distribution between piles
V_{cp}, V_r = Nominal and factored concrete shear capacity of C.I.P. pile respectively
α_t = coefficient of thermal expansion for superstructure
x_{TO} = distance from seat to point of thermal origin
Δ_{temp} = displacement demand from temperature effects
Δ_{tmin} = minimum displacement demand from temperature effects
Δ_{ot} = Movement attributed to effects other than seismic
S_k = skew of bent measured from a line normal to the centerline of bridge
Δeq = seimic displacement demand in the bridge longitudinal direction between expansion joints
Min_Seat_Width = minimum seat width required at expansion bents

Piles: Compression and Tension Checks (3x3 c.i.p. pile arrangement)

$L_{foot} := 13.5ft$

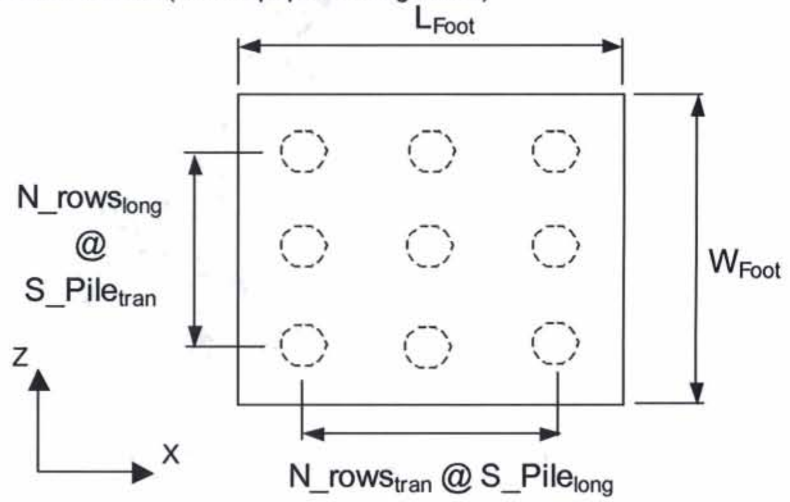
$W_{foot} := 13.5ft$

$N\_rows_{long} := 3$

$S\_Pile_{tran} := 5ft$

$N\_rows_{tran} := 3$

$S\_Pile_{long} := 5ft$



$Pile\_Dia := 20in$ $\quad A_{pile\_st} := 3.53in^2$ $\quad A_{shell} := 23.12in^2$ $\quad$ *Assume Shell Fy = 45 ksi*

Note: For pushover analysis 14" piles would have been adequate for tension, compression, and shear. 20" piles were used to match the design for LFD.

$I_{pgx} := 6 \cdot S\_Pile_{tran}{}^2$ $\qquad I_{pgx} = 2.16 \times 10^4\, in^2$

$I_{pgz} := 6 \cdot S\_Pile_{long}{}^2$ $\qquad I_{pgz} = 2.16 \times 10^4\, in^2$

Left Footing | **Right Footing**

$M_{pL} = 1.649 \times 10^3\, k \cdot ft$ $\qquad M_{pR} = 1.989 \times 10^3\, k \cdot ft$

$P_L = 318.529\, k$ $\qquad P_R = 641.471\, k$

$DL\_Sub := \left[B \cdot (2 \cdot Cant + 2 \cdot S_{col}) \cdot Bm\_Ht + N_{col} \cdot Col\_Ht \cdot A_g\right] \cdot 0.15 \dfrac{k}{ft^3}$ $\qquad DL\_Sub = 223.502\, k$

$P_{TOF\_L} := \dfrac{DL\_Sub}{N_{col}} + P_L$ $\qquad P_{TOF\_L} = 393.03\, k$

$P_{BOF\_L} := P_{TOF\_L} + D_{foot} \cdot L_{foot} \cdot W_{foot} \cdot 0.15kcf$ $\qquad P_{BOF\_L} = 516.049\, k$

$P_{BOF\_R} := P_R + P_{BOF\_L} - P_L$ $\qquad P_{BOF\_R} = 838.99\, k$

Use Linear Distribution of Forces?

$$\text{ForceDistCheck} := \begin{cases} \text{"O.K."} & \text{if } \dfrac{L_{foot} - D}{2D_{foot}} \le 2.5 \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{\text{ForceDistCheck} = \text{"O.K."}}$

Note: Per LRFD Seismic 6.4.2 the linear distribution of forces may be considered when standard size piles are used (Diameter <= 16"). Assume procedure is adequate for 20" piles.

$$R_{min1} := \frac{P_{BOF\_L}}{N\_rows_{long} \cdot N\_rows_{tran}} - M_{pL} \cdot \frac{S\_Pile_{long}}{I_{pgz}}$$

...Plastic Moment about Z-axis

$R_{min1} = 2.372\,k$

$$R_{max1} := \frac{P_{BOF\_R}}{N\_rows_{long} \cdot N\_rows_{tran}} + M_{pR} \cdot \frac{S\_Pile_{long}}{I_{pgz}}$$

$R_{max1} = 159.521\,k$

$$R_{min2} := \frac{P_{BOF\_L}}{N\_rows_{long} \cdot N\_rows_{tran}} - M_{pL} \cdot \frac{S\_Pile_{tran}}{I_{pgx}}$$

...Plastic Moment about X-axis

$R_{min2} = 2.372\,k$

$$R_{max2} := \frac{P_{BOF\_R}}{N\_rows_{long} \cdot N\_rows_{tran}} + M_{pR} \cdot \frac{S\_Pile_{tran}}{I_{pgx}}$$

$R_{max2} = 159.521\,k$

$R_{min} := \min(R_{min1}, R_{min2})$

$R_{max} := \max(R_{max1}, R_{max2})$

$$T_{pile} := \begin{cases} 20k & \text{if } Pile\_Dia = 14in \\ 26.67k & \text{if } Pile\_Dia = 20in \\ 32k & \text{if } Pile\_Dia = 24in \end{cases}$$

$T_{pile} = 26.67\,k$

Check whether shear cone failure requires overlap

$$\text{Overlap\_tran} := \begin{cases} \text{"Yes"} & \text{if } Pile\_Dia + 36in > S\_Pile_{tran} \\ \text{"No"} & \text{otherwise} \end{cases}$$

Overlap_tran = "No"

$$\text{Overlap\_long} := \begin{cases} \text{"Yes"} & \text{if } Pile\_Dia + 36in > S\_Pile_{long} \\ \text{"No"} & \text{otherwise} \end{cases}$$

Overlap_long = "No"

$$Radius1 := \begin{cases} \dfrac{Pile\_Dia}{2} + 18in & \text{if } Overlap\_tran = \text{"No"} \\ \dfrac{S\_Pile_{tran}}{2} & \text{otherwise} \end{cases}$$

$Radius1 = 28\,in$

$$Radius2 := \frac{W_{foot}}{2} - S\_Pile_{tran}$$

$Radius2 = 21\,in$

$$Radius3 := \begin{cases} \dfrac{Pile\_Dia}{2} + 18in & \text{if } Overlap\_long = \text{"No"} \\ \dfrac{S\_Pile_{long}}{2} & \text{otherwise} \end{cases}$$

$Radius3 = 28\,in$

$$Radius4 := \frac{L_{foot}}{2} - S\_Pile_{long}$$

$Radius4 = 21\,in$

$$Radius := \min(Radius1, Radius2, Radius3, Radius4)$$

$Radius = 21\,in$

$$Slope\_Length := \sqrt{2} \cdot Radius$$

$Slope\_Length = 29.698\,in$

$$A_{surface} := \pi \cdot Slope\_Length \cdot \left(\frac{Pile\_Dia}{2} + Radius \right)$$

$A_{surface} = 2.892 \times 10^3\,in^2$

$$SC\_V_c := 0.0316 \cdot 2 \cdot \sqrt{f_{ce} \cdot ksi} \cdot A_{surface}$$

$SC\_V_c = 408.741\,k$

$$T_{shear} := \frac{SC\_V_c}{3}$$

$T_{shear} = 136.247\,k$

$T_{friction} := 40k$

Note: The friction capacity would normally be calculated by SPILE or LPILE. Friction capacity was not determined for this example (placeholder value).

$$T_{all} := \min(T_{pile}, T_{shear}, T_{friction})$$

$T_{all} = 26.67\,k$

$$Uplift\_Check := \begin{cases} \text{"O.K."} & \text{if } R_{min} > -T_{all} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{Uplift\_Check = \text{"O.K."}}$

$$A_{g\_pile} := \pi \cdot \frac{Pile\_Dia^2}{4}$$

$C_{friction} := 1.5 \cdot 120k$ *Assume pile has 60 ton capacity with a 50% overstress allowed for instantaneous loading*

$C_{friction} = 180\,k$

$$\text{Compression\_Check} := \begin{cases} \text{"O.K."} & \text{if } R_{max} < C_{friction} \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{\text{Compression\_Check} = \text{"O.K."}}$

Piles: Shear Check

Assume that right column always controls.

$V_{pR} = 89.143\,k$

$V_{pile} := \dfrac{V_{pR}}{9}$ $V_{pile} = 9.905\,k$

Calculate shear capacity of each pile, V_n:

Class = "D" Pile_Dia = 1.667 ft $A_{g\_pile} = 314\,in^2$ $A_{e\_pile} := 0.8 A_{g\_pile}$

Tie_Bar := 4 $V_{cp} := 0.0632 \cdot ksi \cdot A_{g\_pile}$ $V_{cp} = 19.855\,k$

$V_r := \phi \cdot V_{cp}$ $V_r = 16.877\,k$

$$\text{Pile\_Shear\_Check} := \begin{cases} \text{"O.K."} & \text{if } V_{pile} \le V_r \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$\boxed{\text{Pile\_Shear\_Check} = \text{"O.K."}}$

Smaller piles may require added shear capacity from reinforcement (i.e., 14" piles are adequate, but not for concrete capacity alone). For this example the concrete shear capacity is adequate to resist the shear force induced with a plastic column moment.

Piles: Details

Pile Length is not considered for this example. A 1/2" thick steel casing is used throughout the pile length. Piles require longitudinal reinforcement to extend half the pile length per LRFD Seismic 8.16.2. Piles are embedded 18" per office practice. Reinforcement includes 8 - #6 bars extended into the footing with hooks clearing the top of footing by 1.5". #4 hoops at 3" centers are extended to the top of hooks and a minimum of 4 ft below the bottom of footing. #4 hoops at 9" centers are then used to extend to half the pile length.

Minimum Seat Length LRFD Seismic 4.12

Note: Expansion joints are located at the abutments. The length between expansion joints is 248'. Fixed joint located at center intermediate bent. Assume 248' expansion length with total temperature range (150 deg. for steel superstructure)

$\alpha_t := 0.0000065 \frac{in}{in \cdot F}$ $\quad$ $Temp\_Range := 150F$ $\quad$ $x_{TO} := 248ft$

$\Delta_{temp} := \alpha_t \cdot Temp\_Range \cdot x_{TO}$ $\quad$ $\Delta_{temp} = 2.902\,in$

$\Delta_{tmin} := x_{TO} \cdot 1 \cdot \frac{in}{100ft}$ $\quad$ $\Delta_{tmin} = 2.48\,in$

$\Delta_{ot} := \max(\Delta_{temp}, \Delta_{tmin})$ $\quad$ $\Delta_{ot} = 2.902\,in$

$S_k := 18.36$

$\Delta_{eq} := \Delta_C$ $\quad$ $\Delta_{eq} = 17.247\,in$

$$Min\_Seat\_Width := \max\left[12 \cdot in, \left(4 \cdot in + \Delta_{ot} + 1.65 \cdot \Delta_{eq}\right) \cdot \frac{1 + S_k^{\,2}}{4000}\right]$$

$\boxed{Min\_Seat\_Width = 12\,in}$

Δeq requires displacement demand in longitudinal direction of bridge. This value is unknown and will be assumed to be equal to the bent longitudinal displacement capacity, Δ_c.

DC Loads
Self Weight Generation Disabled
Traffic Barrier Load Disabled

Span	1	W	8.747e+03	plf	from	0.000 ft	to	86.000 ft	
Span	1	W	9.462e+03	plf	from	86.000 ft	to	124.000 ft	
Span	2	W	9.462e+03	plf	from	0.000 ft	to	38.000 ft	
Span	2	W	8.747e+03	plf	from	38.000 ft	to	124.000 ft	

DW Loads
Utility Load Disabled
Wearing Surface Load Disabled

Live Load Data

Live Load Generation Parameters
Design Tandem : Disabled
Design Truck : Disabled
Dual Truck Train : Disabled
Dual Tandem Train: Disabled
Fatigue Truck : Disabled

Live Load Impact
Truck Loads 33.000%
Lane Loads 0.000%
Fatigue Truck 15.000%

Pedestrian Live Load 0.000e+00 plf

Load Factors

Strength I	DC min	0.900	DC max	1.250	DW min	0.650	DW max	1.500	LL	1.750
Service I	DC	1.000	DW	1.000	LL	1.000				
Service II	DC	1.000	DW	1.000	LL	1.300				
Service III	DC	1.000	DW	1.000	LL	0.800				
Fatigue	DC	0.000	DW	0.000	LL	0.750				

Analysis Results

DC Dead Load

Span	Point	Shear(lbs)	Moment(ft-lbs)
1	0	391.831e+03	0.000e+00
1	1	283.368e+03	4.186e+06
1	2	174.906e+03	7.027e+06
1	3	66.443e+03	8.523e+06
1	4	-42.019e+03	8.675e+06
1	5	-150.482e+03	7.481e+06
1	6	-258.945e+03	4.943e+06
1	7	-367.980e+03	1.059e+06
1	8	-485.309e+03	-4.230e+06
1	9	-602.639e+03	-10.975e+06
1	10	-719.968e+03	-19.176e+06
2	0	719.968e+03	-19.176e+06
2	1	602.639e+03	-10.975e+06
2	2	485.309e+03	-4.230e+06
2	3	367.980e+03	1.059e+06
2	4	258.945e+03	4.943e+06
2	5	150.482e+03	7.481e+06
2	6	42.019e+03	8.675e+06
2	7	-66.443e+03	8.523e+06
2	8	-174.906e+03	7.027e+06

DL Reaction Total

$= 2(720^k) = 1440^k$

2	9	-283.368e+03	4.186e+06
2	10	-391.831e+03	0.000e+00

XTRACT Section Report

Section Name: Section6

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MoDOT
3/14/2006
Stoddard Co.
Bent2
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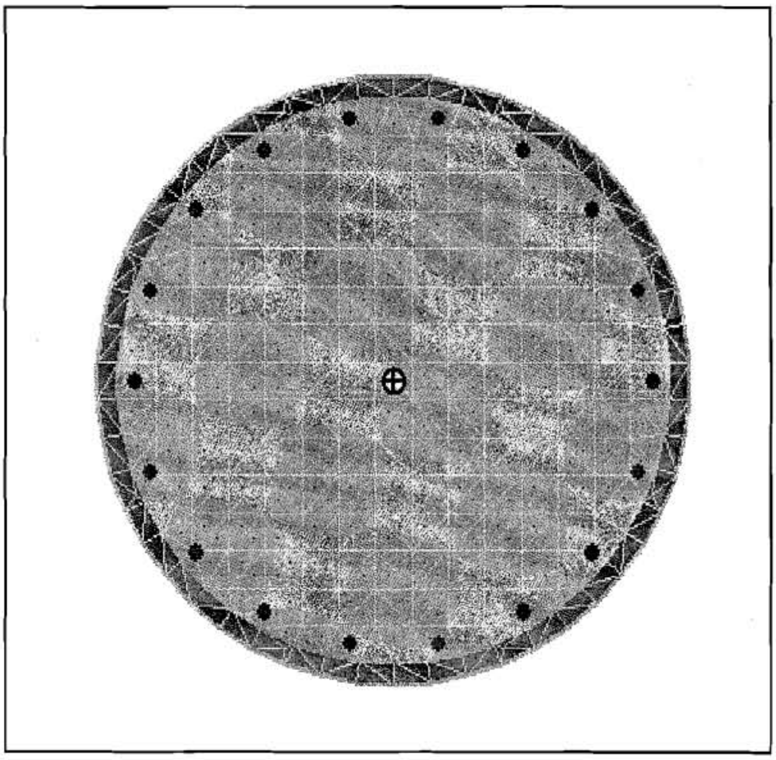
Section Details:

X Centroid:	-.9109E-16 ft
Y Centroid:	-.1857E-16 ft
Section Area:	9.596 ft^2
I gross about X:	7.420 ft^4
I gross about Y:	7.420 ft^4
Reinforcing Bar Area:	98.17E-3 ft^2
Percent Longitudinal Steel:	1.023 %
Overall Width:	3.493 ft
Overall Height:	3.500 ft
Number of Fibers:	492
Number of Bars:	18
Number of Materials:	3



Material Types and Names:

Unconfined Concrete:	Unconfined1
Strain Hardening Steel:	Steel1
Confined Concrete:	Confined7

XTRACT Analysis Report

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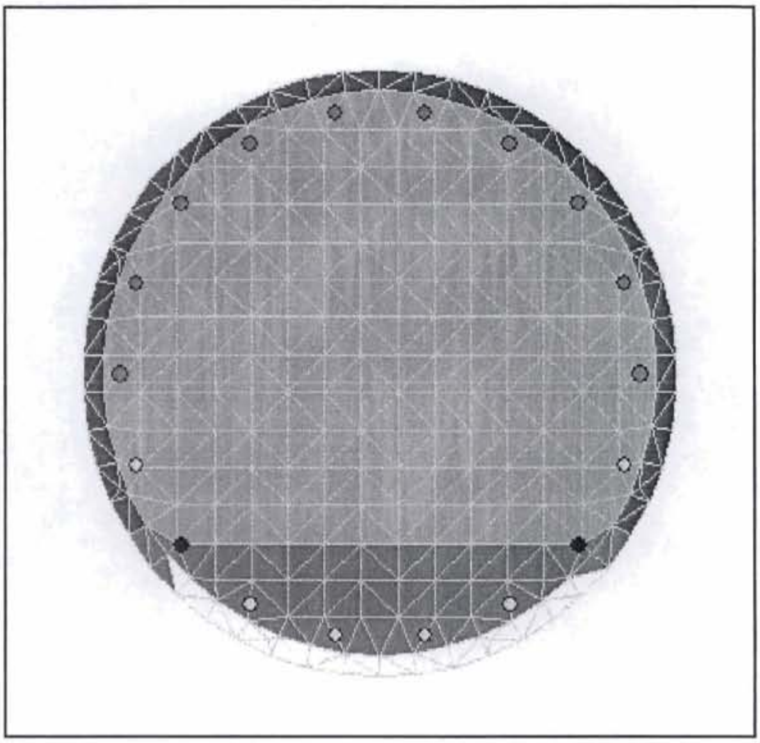
Section Name:	Section6
Loading Name:	MC1
Analysis Type:	Moment Curvature

Section Details:

X Centroid:	-.9109E-16 ft
Y Centroid:	-.1857E-16 ft
Section Area:	9.596 ft^2

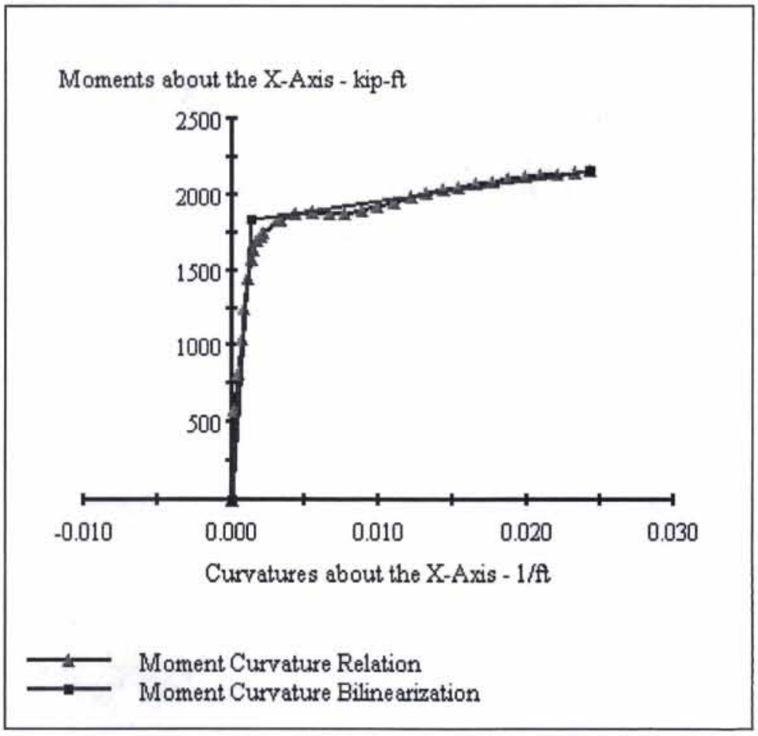


Loading Details:

Constant Load - P:	480.0 kips
Incrementing Loads:	Mxx Only
Number of Points:	30
Analysis Strategy:	Displacement Control

Analysis Results:

Failing Material:	Steel1
Failure Strain:	60.00E-3 Tension
Curvature at Initial Load:	-.2060E-21 1/ft
Curvature at First Yield:	1.083E-3 1/ft
Ultimate Curvature:	24.35E-3 1/ft
Moment at First Yield:	1449 kip-ft
Ultimate Moment:	2155 kip-ft
Centroid Strain at Yield:	.6431E-3 Ten
Centroid Strain at Ultimate:	23.28E-3 Ten
N.A. at First Yield:	.5940 ft
N.A. at Ultimate:	.9559 ft
Energy per Length:	47.05 kips
Effective Yield Curvature:	1.368E-3 1/ft
Effective Yield Moment:	1830 kip-ft
Over Strength Factor:	1.177
Plastic Rotation Capacity:	59.99E-3 rad
EI Effective:	1.338E+6 kip-ft^2
Yield EI Effective:	14.13E+3 kip-ft^2
Bilinear Harding Slope:	1.056 %
Curvature Ductility:	17.81



XTRACT Analysis Report

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3/14/2006
Stoddard Co.
Bent2
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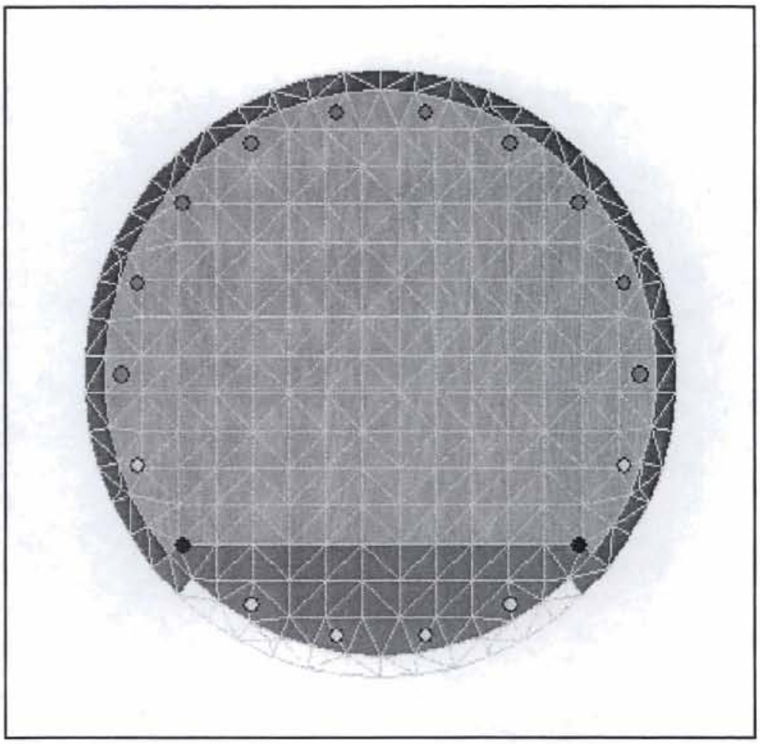
Section Name:	Section6
Loading Name:	MC2 (Left Column)
Analysis Type:	Moment Curvature

Section Details:

X Centroid:	-.9109E-16 ft
Y Centroid:	-.1857E-16 ft
Section Area:	9.596 ft^2

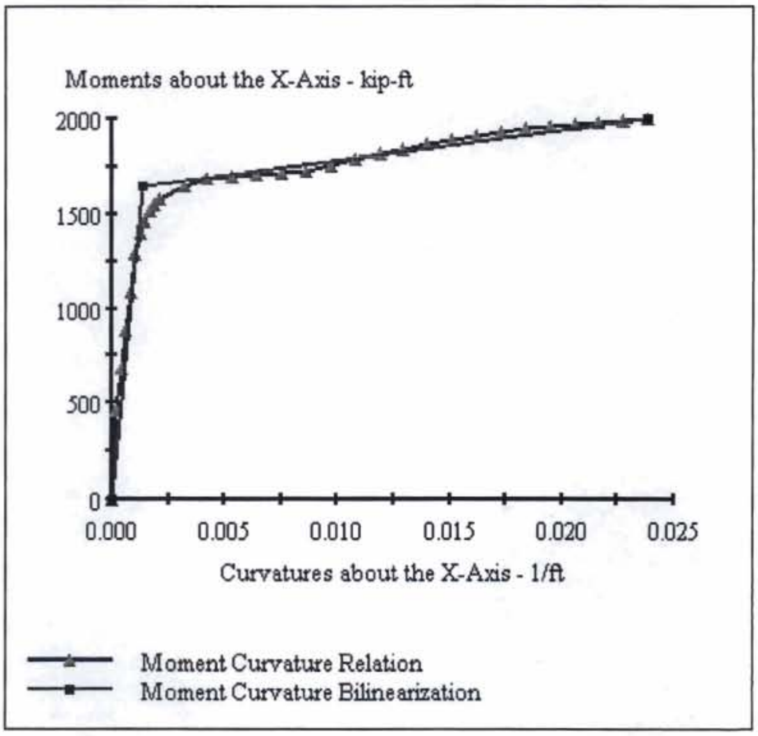


Loading Details:

Constant Load - P:	319.0 kips
Incrementing Loads:	Mxx Only
Number of Points:	30
Analysis Strategy:	Displacement Control

Analysis Results:

Failing Material:	Steel1
Failure Strain:	60.00E-3 Tension
Curvature at Initial Load:	.3438E-20 1/ft
Curvature at First Yield:	1.042E-3 1/ft
Ultimate Curvature:	23.83E-3 1/ft
Moment at First Yield:	1285 kip-ft
Ultimate Moment:	2000 kip-ft
Centroid Strain at Yield:	.7038E-3 Ten
Centroid Strain at Ultimate:	24.06E-3 Ten
N.A. at First Yield:	.6751 ft
N.A. at Ultimate:	1.009 ft
Energy per Length:	42.14 kips
Effective Yield Curvature:	1.338E-3 1/ft
Effective Yield Moment:	1649 kip-ft
Over Strength Factor:	1.213
Plastic Rotation Capacity:	58.72E-3 rad
EI Effective:	1.232E+6 kip-ft^2
Yield EI Effective:	15.60E+3 kip-ft^2
Bilinear Harding Slope:	1.266 %
Curvature Ductility:	17.81



XTRACT Analysis Report

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Stoddard Co.
Bent2
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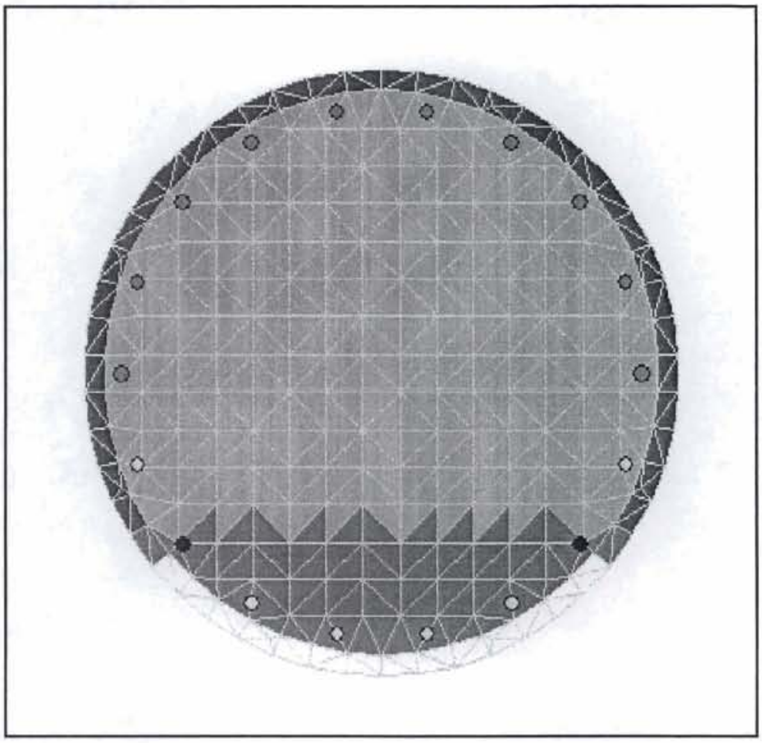
Section Name:	Section6
Loading Name:	MC3 (Right Column)
Analysis Type:	Moment Curvature

Section Details:

X Centroid:	-.9109E-16 ft
Y Centroid:	-.1857E-16 ft
Section Area:	9.596 ft^2

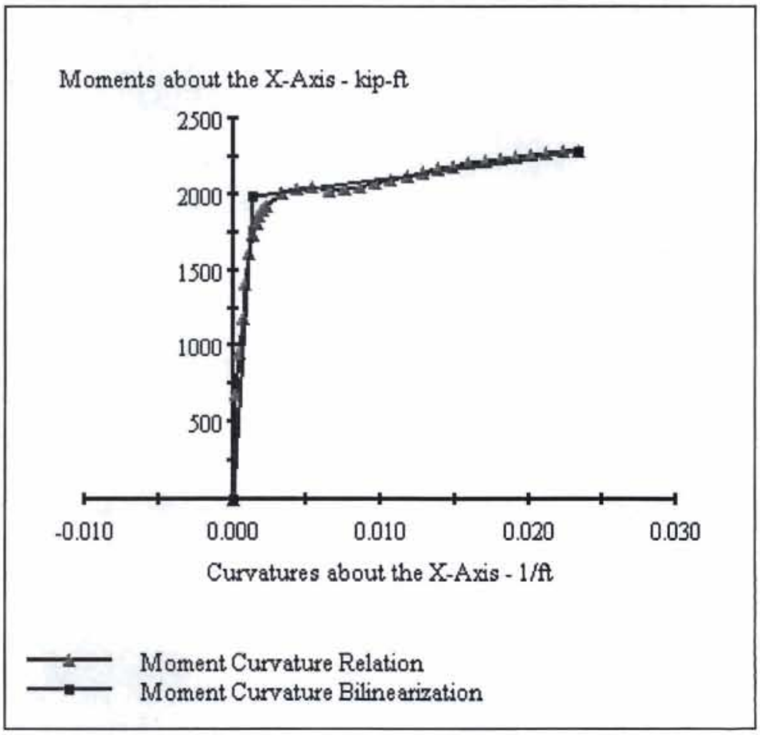


Loading Details:

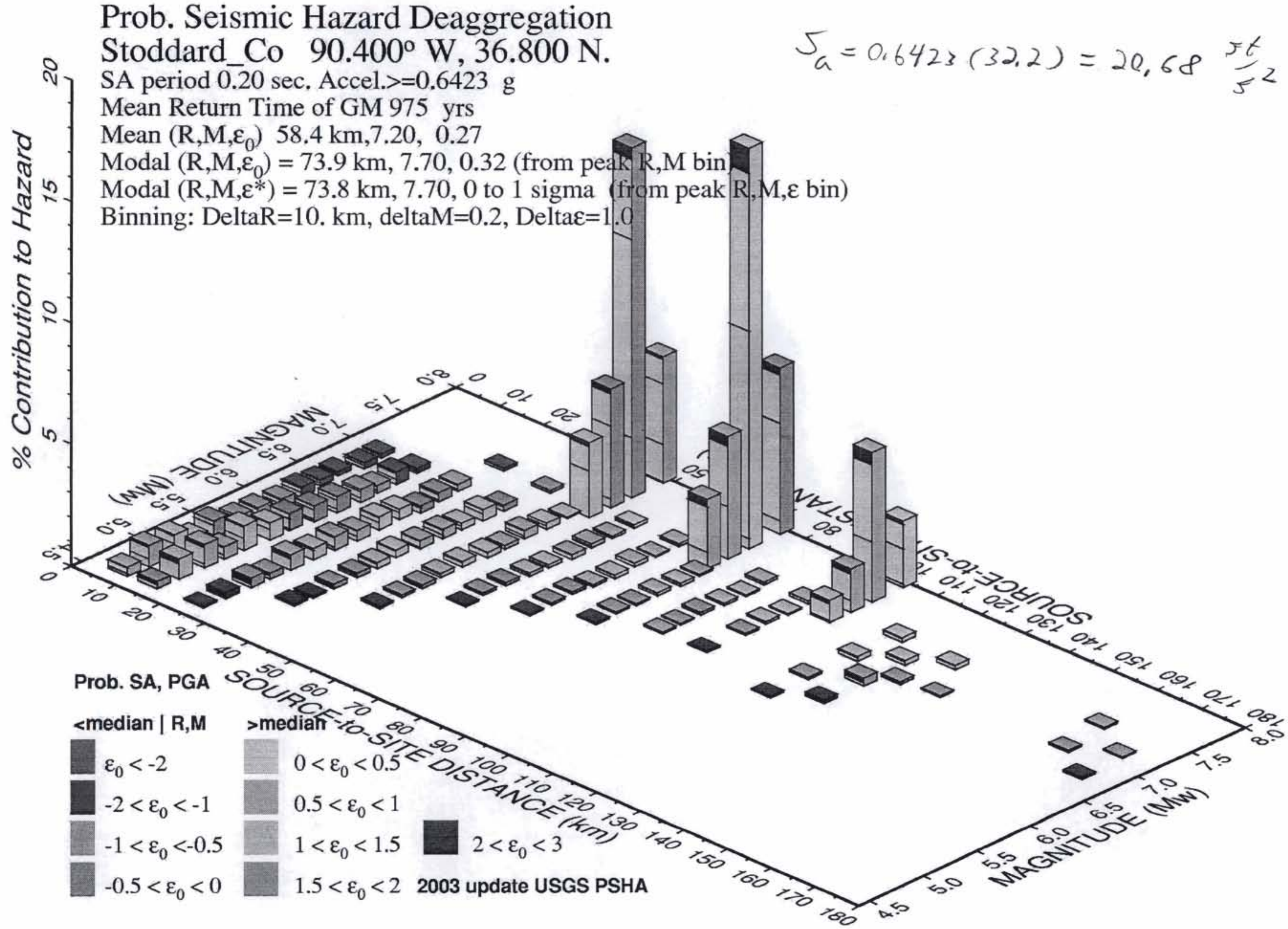
Constant Load - P:	641.0 kips
Incrementing Loads:	Mxx Only
Number of Points:	30
Analysis Strategy:	Displacement Control

Analysis Results:

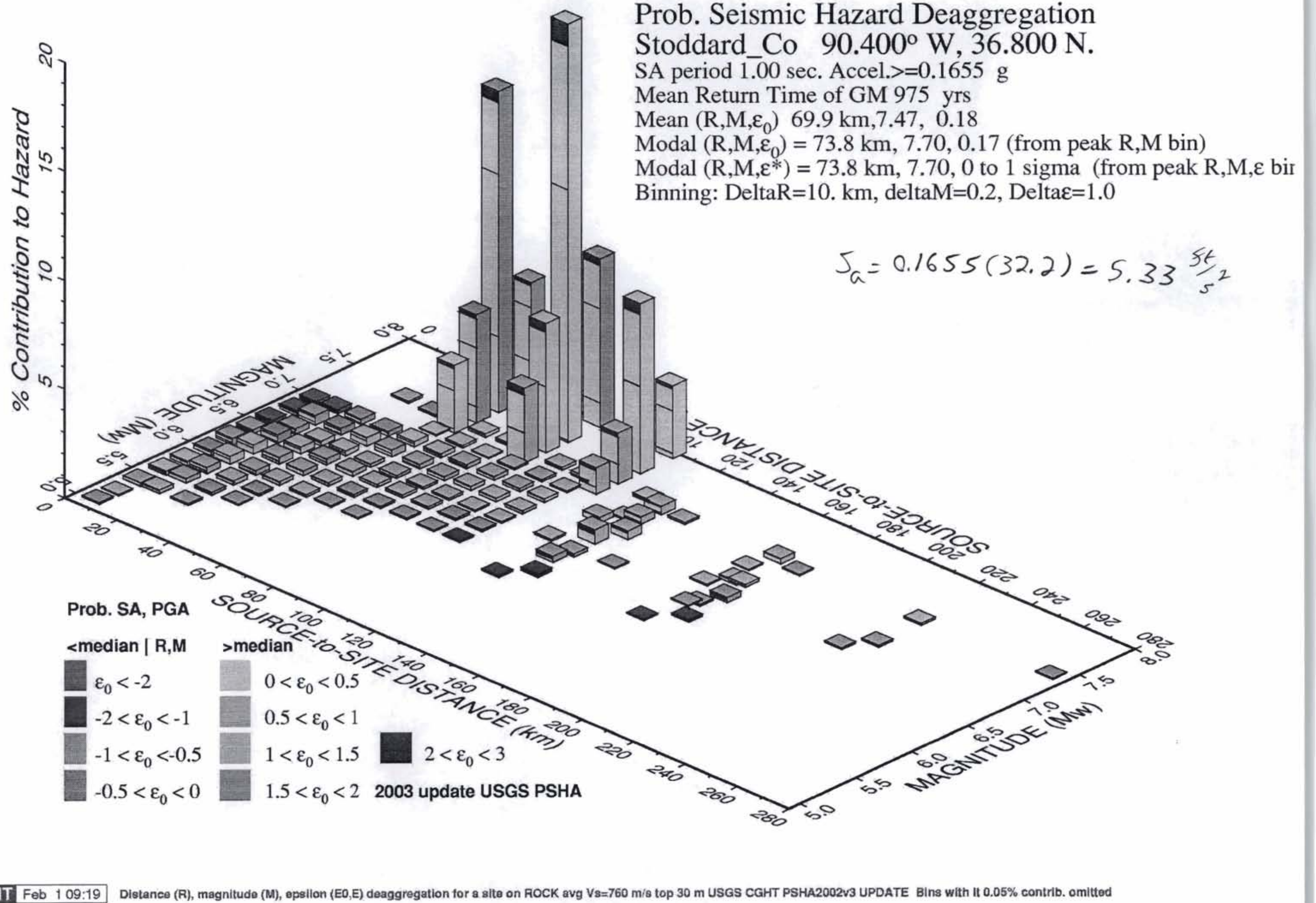
Failing Material:	Confined7
Failure Strain:	15.42E-3 Compression
Curvature at Initial Load:	-.1174E-20 1/ft
Curvature at First Yield:	1.122E-3 1/ft
Ultimate Curvature:	23.40E-3 1/ft
Moment at First Yield:	1608 kip-ft
Ultimate Moment:	2290 kip-ft
Centroid Strain at Yield:	.5843E-3 Ten
Centroid Strain at Ultimate:	20.81E-3 Ten
N.A. at First Yield:	.5208 ft
N.A. at Ultimate:	.8893 ft
Energy per Length:	48.49 kips
Effective Yield Curvature:	1.388E-3 1/ft
Effective Yield Moment:	1989 kip-ft
Over Strength Factor:	1.151
Plastic Rotation Capacity:	57.46E-3 rad
EI Effective:	1.433E+6 kip-ft^2
Yield EI Effective:	13.67E+3 kip-ft^2
Bilinear Harding Slope:	.9533 %
Curvature Ductility:	16.87



$S_a = 0.6423\,(32.2) = 20.68\ \frac{ft}{s^2}$



GMT Feb 1 09:16 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on ROCK avg Vs=760 m/s top 30 m USGS CGHT PSHA2002v3 UPDATE Bins with lt 0.05% contrib. omitted



$S_a = 0.1655(32.2) = 5.33 \ \frac{ft}{s^2}$

Seismic Design Response Spectrum Input Req'd

A7029 - (pin-fix) clear height

Design Seismic Event - 5% Probability of Occurrence in 50 years

Site Classification

Site class = **D** Based on site condition

Bridge location

Latitude = 36.8 deg.

Longitude = -90.4 deg. (use -ive value)

As per bridge location & USGS map

Ss = 20.680 0.2-second period spectral acceleration

S_1 = 5.330 1-second period spectral acceleration

Fa = 1.00 Read from Table 3.4.2.3-1 based on site class & Ss value

Fv = 1.50 Read from Table 3.4.2.3-1 based on site class & S_1 value

$S_{DS} = Fa*Ss =$ 20.68

$S_{D1} = Fv*S_1 =$ 8.00

SDC = **D** Based on LRFD Table 3.5-1

<u>RESPONSE SPECTRUM DEVELOPMENT</u>

If T <= To $Sa = 0.6(S_{DS}/T_0)*T + 0.4*S_{DS}$

If To <= T <= Ts $Sa = S_{DS}$

If T > Ts $Sa = S_{D1}/T$

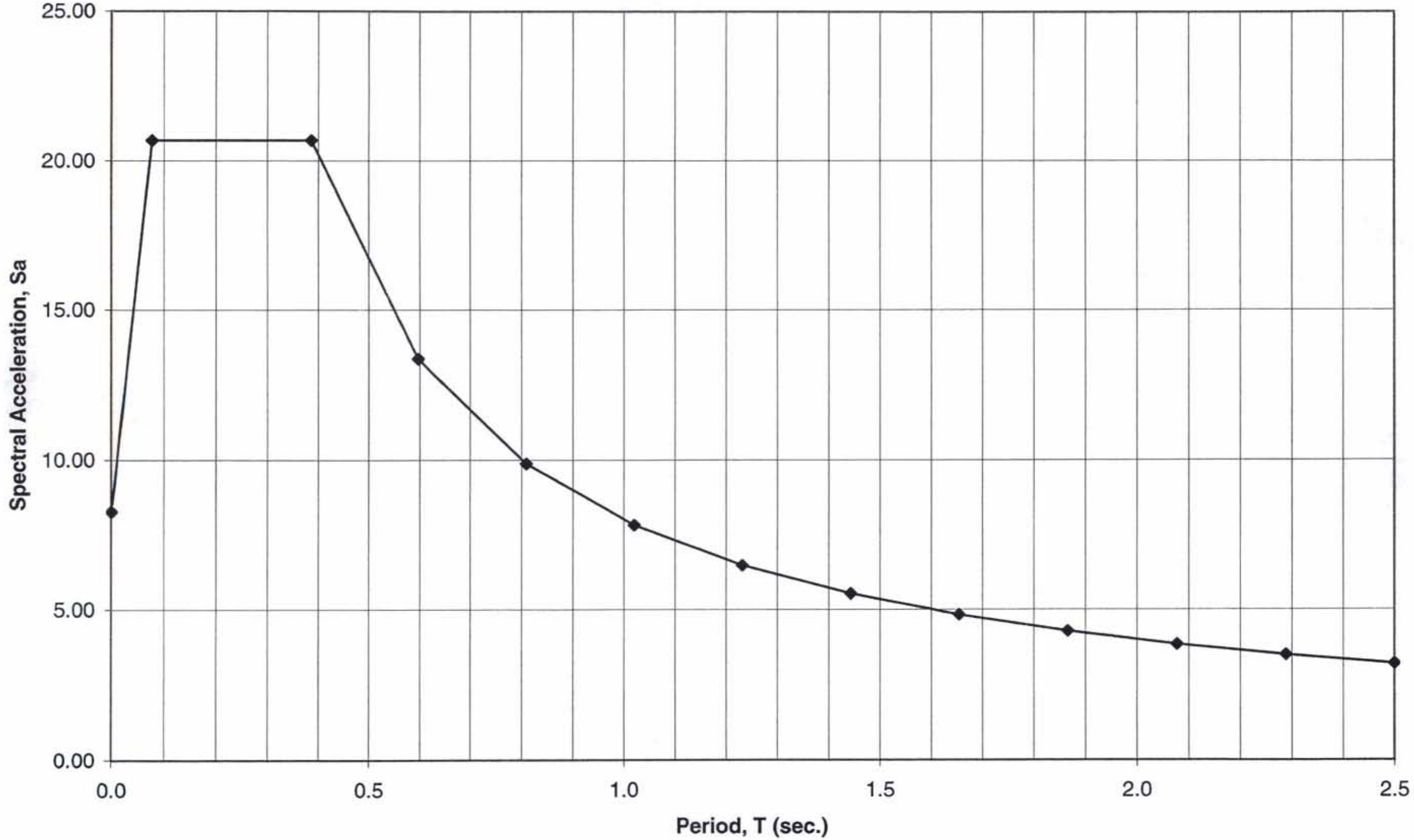
	T	Sa	
	0	8.27	$= 0.4S_{DS}$
$T_0 = 0.2*Ts =$	0.08	20.68	$= S_{DS}$
$Ts = S_{D1}/S_{DS} =$	0.39	20.68	$= S_{DS}$
	0.60	13.37	
	0.81	9.88	
	1.02	7.83	
	1.23	6.49	
	1.44	5.54	
	1.65	4.83	
	1.87	4.28	
	2.08	3.85	
	2.29	3.49	
	2.50	3.20	

For a known "T":

T = **1.28**

Sa = **<u>6.26</u>**

Response Spectrum



(P/Pu)^2 + 14.19E+3(P/Pu)^3

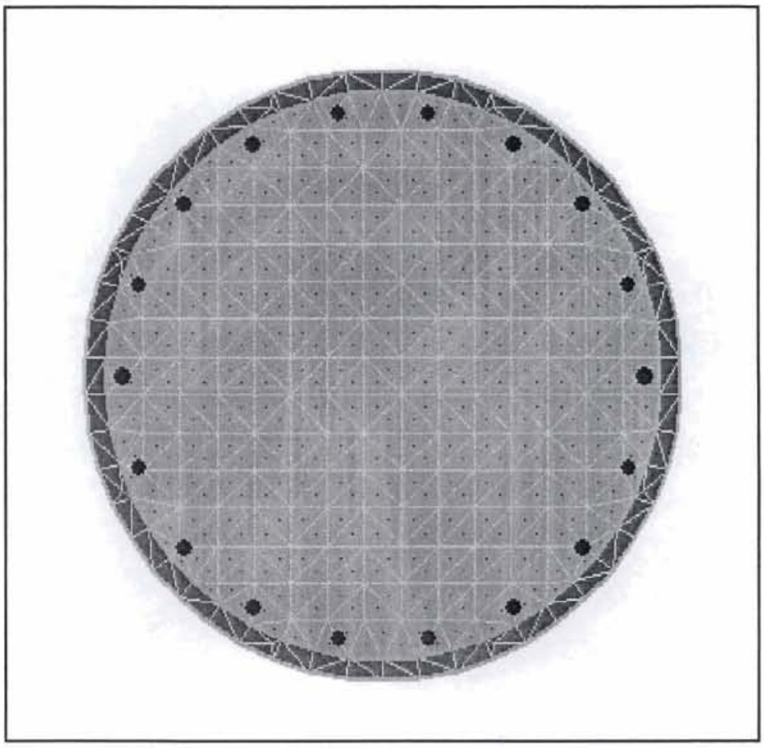
XTRACT Analysis Report

D. Kemna
MoDOT
3/14/2006
Stoddard Co.
Bent2
Page __ of __

Section Name:	Section6
Loading Name:	LC1
Analysis Type:	PM Interaction

Section Details:

X Centroid:	-.9109E-16 ft
Y Centroid:	-.1857E-16 ft
Section Area:	9.596 ft^2

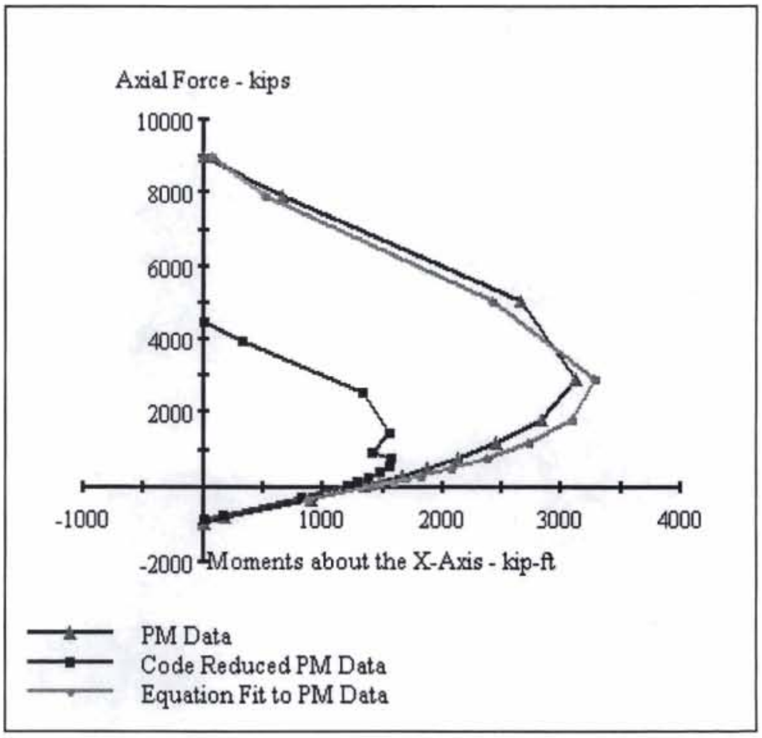


Loading Details:

Angle of Loading:	0 deg
Number of Points:	20
Min. Unconfined1 Strain:	3.000E-3 Comp
Max. Unconfined1 Strain:	1.0000 Ten
Min. Steel1 Strain:	15.00E-3 Comp
Max. Steel1 Strain:	15.00E-3 Ten
Min. Confined7 Strain:	5.220E-3 Comp
Max. Confined7 Strain:	1.0000 Ten

Analysis Results:

Max. Compression Load:	8985 kips
Max. Tension Load:	-933.1 kips
Maximum Moment:	3121 kip-ft
P at Max. Moment:	2891 kips
Minimum Moment:	-.2678E-12 kip-ft
P at Min. Moment:	8985 kips
Moment (Mxx) at P=0:	1317 kip-ft
Max. Code Comp. Load:	4493 kips
Max. Code Ten. Load:	-839.7 kips
Maximum Code Moment:	1567 kip-ft
P at Max. Code Moment:	749.7 kips
Minimum Code Moment:	-.1339E-12 kip-ft
P at Min. Code Moment:	4493 kips
PM Interaction Equation:	Units in kip-ft



XTRACT Analysis Report

D. Kemna
MoDOT
3/14/2006
Stoddard Co.
Bent2
Page __ of __

Section Name: Section6
Loading Name: LC1
Analysis Type: PM Interaction

P (kips)	Mxx (kip-ft)
8985	-.2678E-12
7903	656.1
5041	2666
2891	3121
1792	2832
1174	2453
772.6	2134
496.7	1869
299.4	1664
133.2	1480
19.17	1342
-312.0	907.6
-809.2	182.0
-933.1	.2898E-12
-933.1	.2898E-12
-933.1	.2898E-12
-933.1	.2898E-12
-933.1	.2898E-12
-933.1	.2898E-12
-933.1	.2898E-12
-933.1	.2898E-12

```
# CAPP Generated Input file
# Created = 3/14/2006

UNITS
     FORCE=Kips
     LENGTH=Feet

JOINT
     NUM=1  X=-17  Y=0
     NUM=2  X=-17  Y=1.395
     NUM=3  X=-17  Y=20.92
     NUM=4  X=-17  Y=22.31
     NUM=5  X=-17  Y=24.56
     NUM=6  X=0  Y=0
     NUM=7  X=0  Y=1.395
     NUM=8  X=0  Y=20.92
     NUM=9  X=0  Y=22.31
     NUM=10  X=0  Y=24.56
     NUM=11  X=17  Y=0
     NUM=12  X=17  Y=1.395
     NUM=13  X=17  Y=20.92
     NUM=14  X=17  Y=22.31
     NUM=15  X=17  Y=24.56
     NUM=16  X=-23.5  Y=24.56
     NUM=17  X=23.5  Y=24.56

RESTRAINT
     NUM=1 DOF=U1,U2
     NUM=6 DOF=U1,U2
     NUM=11 DOF=U1,U2

MATERIAL
     NAME=Concrete  TYPE=Concrete  E=586.7E+3  W=0.15

SECTION
     NAME=Col1  TYPE=User_Defined  MAT=Concrete  I=2.101  A=9.618  DESCRIPTION=Cracked left column
     NAME=Col2  TYPE=User_Defined  MAT=Concrete  I=2.28  A=9.618  DESCRIPTION=Cracked center column
     NAME=Col3  TYPE=User_Defined  MAT=Concrete  I=2.442  A=9.618  DESCRIPTION=Cracked right column
     NAME=Beam  TYPE=User_Defined  MAT=Concrete  I=7.986  A=18  DESCRIPTION=Cracked beam

ELEMENT
     NAME=LeftCol  TYPE=Elastic_Beam_Column  SEC=Col1
     NAME=CentCol  TYPE=Elastic_Beam_Column  SEC=Col2
     NAME=RtCol  TYPE=Elastic_Beam_Column  SEC=Col3
     NAME=CapBeam  TYPE=Elastic_Beam_Column  SEC=Beam
     NAME=Rigid  TYPE=Rigid_Link

HINGE
     NAME=Hinge1  TYPE=Interaction_Hinge  ROTCAP=59.99E-3  CLIMIT=8985  TLIMIT=-933.1  PU=8985  HINGE_MODE=2
          DATA  P=-933.1  M=.1E-9
          DATA  P=-809.2  M=182
          DATA  P=-312  M=907.6
          DATA  P=19.17  M=1342
          DATA  P=133.2  M=1480
          DATA  P=299.4  M=1664
          DATA  P=496.7  M=1869
          DATA  P=772.6  M=2134
          DATA  P=1174  M=2453
          DATA  P=1792  M=2832
          DATA  P=2891  M=3121
          DATA  P=5041  M=2666
          DATA  P=7903  M=656.1
          DATA  P=8985  M=.1E-9
     NAME=Hinge2  TYPE=Interaction_Hinge  ROTCAP=58.72E-3  CLIMIT=8985  TLIMIT=-933.1  PU=8985  HINGE_MODE=2
          DATA  P=-933.1  M=.1E-9
          DATA  P=-809.2  M=182
          DATA  P=-312  M=907.6
          DATA  P=19.7  M=1342
          DATA  P=133.2  M=1480
          DATA  P=299.4  M=1664
          DATA  P=496.7  M=1869
          DATA  P=772.6  M=2134
          DATA  P=1174  M=2453
          DATA  P=1792  M=2832
          DATA  P=2891  M=3121
          DATA  P=5041  M=2666
          DATA  P=7903  M=656.1
          DATA  P=8985  M=.1E-9
     NAME=Hinge3  TYPE=Interaction_Hinge  ROTCAP=57.46E-3  CLIMIT=8985  TLIMIT=-933.1  PU=8985  HINGE_MODE=2
          DATA  P=-933.1  M=.1E-9
          DATA  P=-809.2  M=182
          DATA  P=-312  M=907.6
          DATA  P=19.17  M=1342
          DATA  P=133.2  M=1480
          DATA  P=299.4  M=1664
          DATA  P=496.7  M=1869
          DATA  P=772.6  M=2134
          DATA  P=1174  M=2453
          DATA  P=1792  M=2832
          DATA  P=2891  M=3121
```

```
          DATA  P=5041  M=2666
          DATA  P=7903  M=656.1
          DATA  P=8985  M=.1E-9

MEMBER
     NUM=1  ELEM=LeftCol  INODE=1  JNODE=2
     NUM=2  ELEM=LeftCol  INODE=2  JNODE=3
     NUM=3  ELEM=LeftCol  INODE=3  JNODE=4  IHINGE=Hinge1
     NUM=4  ELEM=Rigid  INODE=4  JNODE=5
     NUM=5  ELEM=CentCol  INODE=6  JNODE=7
     NUM=6  ELEM=CentCol  INODE=7  JNODE=8
     NUM=7  ELEM=CentCol  INODE=8  JNODE=9  IHINGE=Hinge2
     NUM=8  ELEM=Rigid  INODE=9  JNODE=10
     NUM=9  ELEM=RtCol  INODE=11  JNODE=12
     NUM=10  ELEM=RtCol  INODE=12  JNODE=13
     NUM=11  ELEM=RtCol  INODE=13  JNODE=14  IHINGE=Hinge3
     NUM=12  ELEM=Rigid  INODE=14  JNODE=15
     NUM=13  ELEM=CapBeam  INODE=16  JNODE=5
     NUM=14  ELEM=CapBeam  INODE=5  JNODE=10
     NUM=15  ELEM=CapBeam  INODE=10  JNODE=15
     NUM=16  ELEM=CapBeam  INODE=15  JNODE=17

LOAD
     NAME=Load1  TYPE=Dead_Load
          MEMDATA  NUM=13  IY=-30.64  JY=-30.64
          MEMDATA  NUM=14  IY=-30.64  JY=-30.64
          MEMDATA  NUM=15  IY=-30.64  JY=-30.64
          MEMDATA  NUM=16  IY=-30.64  JY=-30.64
     NAME=Load2  TYPE=Push_Load
          NODEDATA  NUM=16  X=10

COMBO
     NAME=Combo1
          LOAD=Load1  SF=1
          LOAD=Load2  SF=1

GRIDLINES

PROJECT_PROPERTIES
     JOB_NAME=A7029

END
```

CAPP Analysis Report

D. Kemna
MoDOT
3/14/2006
A7029

Loading Name: Combo1

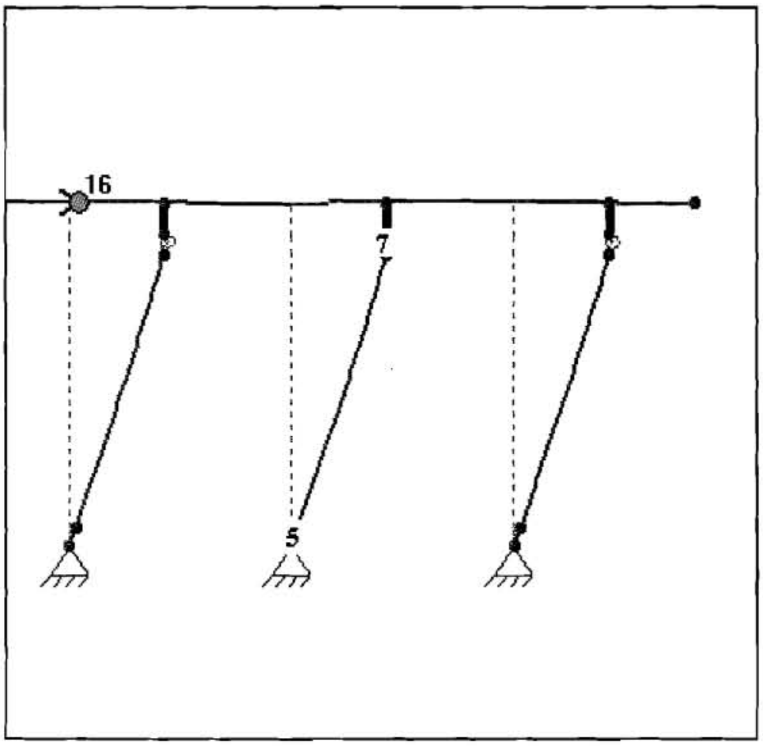
Report Type: Push Over Analysis Summary

Comments:

Page __ of __

Model Details:

Nmber of Members/Nodes:	16 Members, 17 Nodes
Overall Width:	47 ft
Overall Height:	24.56 ft



Loading Details:

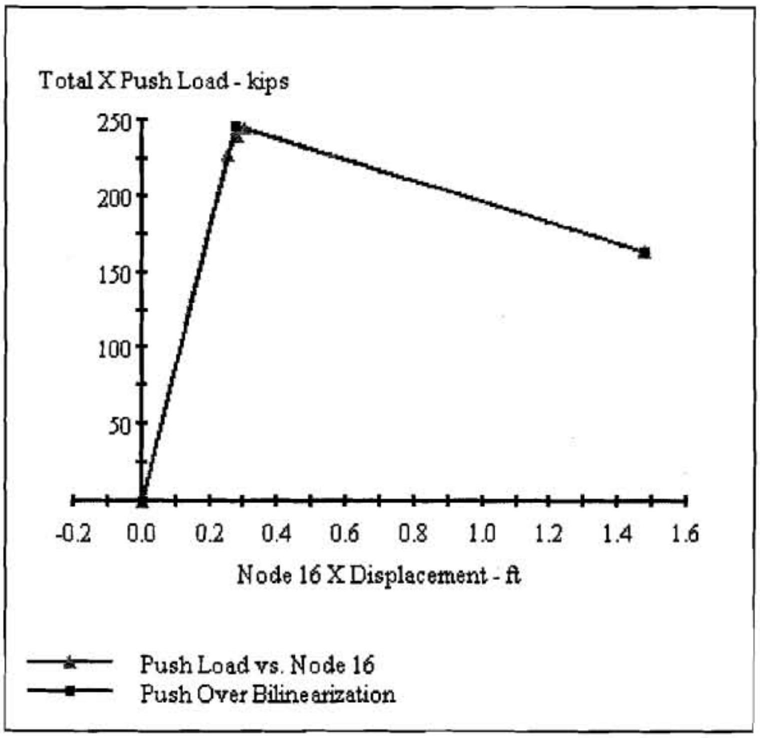
Non-Push Load Combo:	1(Load1)
Push Load Case:	Load2
Num. Loads in Push Case:	1 in X Dir., 0 in Y Dir.
P-Delta Effects Included:	Yes

Termination Details:

Analysis Termination:	Cap. Reached in Mem. 7
Member Element Type:	CentCol - Beam Column
Termination Cause:	Interact. Hinge - Hinge2
Hinge2 :	58.72E-3 rad
Last Hinge Moment:	1878 kips-ft
Mem. Drift at Termination:	0.107%

Analysis Results:

Critical Node (Node Shown):	16
Number of Events:	15
First X Yield Push Load:	227.3 kips
Max X Push Load:	244.6 kips
Last X Push Load:	163.6 kips
X First Yield Displacement:	0.2554 ft
X Ultimate Displacement:	1.478 ft Δ_C
Area Under Push-Disp Curve:	280.2 kips-ft
Effective Yield Disp:	0.2764 ft Δ_Y
Effective Yield Push Load:	246. kips
Eff System Ductility:	5.349
Eff Elastic Stiffness:	889.6 kips/ft
Eff Plastic Stiffness:	68.57 kips/ft
Bilinear Harding Slope:	-7.709 %
Over Strength Factor:	0.6649



$\Delta_D = 3.1'' \Rightarrow \mu_D = \frac{3.1}{3.42''} = 0.91 < 8$ O.K. ; $\Delta_D = 3.1'' < \Delta_C = 17.7''$ O.K.

A7029

PLATE GIRDER
Line Girder : Rating Output : Case Data

Case Data - PLATE GIRDER

AASHTO Specification

Load and Resistance Factor Method
3rd Edition LRFD Bridge Design Specifications
2006 Interims
3rd Edition Append. A and B compactness and moment shifting

Dimensions (additional information available in Dimensions table)

Given dimensions-

Web Depth	60.00 in	60.00 in	60.00 in
Web Thickness	0.75 in	0.75 in	0.75 in
Bearing Stiff. Width	8.00 in	12.00 in	8.00 in
Bearing Stiff. Thickness	0.88 in	1.25 in	0.88 in

Execution Mode

Rate Mode

Geometry

Brace locations

0.00 ft	20.39 ft	40.39 ft	60.39 ft
80.39 ft	100.39 ft	124.00 ft	144.39 ft
164.39 ft	184.39 ft	204.39 ft	224.39 ft
248.00 ft			

Unbraced length of comp. flange at support 2 is 23.61 ft.

Cover plates

No cover plates

Curvature

No curvature

Flange splices

Top flange splice locations

86.00 ft   162.00 ft

Bottom flange splice locations

86.00 ft   162.00 ft

Girder Type

Plate girder
Interior girder

Hinges

No interior hinges

Span lengths

Spans   124.00 ft   124.00 ft

Stiffeners

Bearing stiffeners

Single bearing stiffeners each side

Longitudinal stiffeners

No longitudinal stiffener

Transverse stiffeners

No transverse stiffeners

Web haunches

No web haunches

Web splices

Web splice locations 86.00 ft 162.00 ft

Fatigue

Average Single Lane Daily Truck Traffic: 3200
AWS minimum welds
Fatigue life: 75
Fatigue stress category B flange splices

Loading

Composite region for composite loading- 0. - 248.00 ft
Reaction skew from tangent in degrees
30.00 30.00 30.00
~~Lane fraction for strength limit state~~

To get lane Reactions

Shear	1.0000	1.0000	1.0000	1.0000	1.0000
	1.0000	1.0000	1.0000	1.0000	1.0000
	1.0000	1.0000	1.0000	1.0000	1.0000
	1.0000	1.0000	1.0000	1.0000	~~1.0000~~
	~~1.0000~~				
Moment	0.8000	0.8000	0.8000	0.8000	0.8000
	0.8000	0.8000	0.8000	0.8000	0.8000
	0.8000	0.8000	0.8000	0.8000	0.8000
	0.8000	0.8000	0.8000	0.8000	0.8000
	0.8000				

Lane fraction for fatigue limit state

Shear	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000				
Moment	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000	0.6000	0.6000	0.6000	0.6000
	0.6000				

Lane fraction for live-load deflection
0.5000
Loaded lanes 4
Influence lines not displayed
Unshored construction
Tenth pt values of distributed dead load carried by steel alone

1.38 k/ft	1.38 k/ft	1.38 k/ft	1.38 k/ft	1.38 k/ft
1.38 k/ft	1.38 k/ft	1.54 k/ft	1.54 k/ft	1.54 k/ft
1.54 k/ft	1.54 k/ft	1.54 k/ft	1.54 k/ft	1.38 k/ft
1.38 k/ft	1.38 k/ft	1.38 k/ft	1.38 k/ft	1.38 k/ft
1.38 k/ft				

Steel wt in addition to cross section included in dist dead load

0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft
0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft
0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft
0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft	0.100 k/ft
0.100 k/ft				

Superimposed dead load

0.14 k/ft	0.14 k/ft	0.14 k/ft	0.14 k
0.14 k/ft	0.14 k/ft	0.14 k/ft	0.14 k
0.14 k/ft	0.14 k/ft	0.14 k/ft	0.14 k
0.14 k/ft	0.14 k/ft	0.14 k/ft	0.14 k
0.14 k/ft	0.14 k/ft	0.14 k/ft	0.14 k
0.14 k/ft			

```
   Wearing surface dead load
                                          0.31 k/ft     0.31 k/ft     0.31 k/ft     0.31 k
                                          0.31 k/ft     0.31 k/ft     0.31 k/ft     0.31 k
                                          0.31 k/ft     0.31 k/ft     0.31 k/ft     0.31 k
                                          0.31 k/ft     0.31 k/ft     0.31 k/ft     0.31 k
                                          0.31 k/ft     0.31 k/ft     0.31 k/ft     0.31 k
                                          0.31 k/ft
   Load Factors
       DC1,DC2          1.250
       DW               1.500
       HL93 LL+I        1.750
   Load Modifiers
     Ductility                 0.95
     Redundancy                0.95
     Operational Importance    1.05

Reactions

   Max unfactored live load+Impact reactions
        120.73 k      240.40 k      120.69 k
   Min unfactored live load+im reactions
        -15.48 k        0.00 k      -15.50 k
   Max unfactored live reactions - No dynamic load allowance
         99.26 k      203.36 k       99.23 k
   Min unfactored live reactions - No dynamic load allowance
        -13.00 k        0.00 k      -13.01 k
   Total unfactored dead load DC1+DC2 reactions
         66.14 k      257.05 k       66.14 k
   Total unfactored dead load DW reactions
         13.71 k       48.96 k       13.71 k
   Bearing angle from tangent to web in degrees
         90.00         90.00         90.00

Material


 Concrete

   Concrete strength             4.00 ksi
   Unit wt of concrete            150. lb/cu ft
   Slab T for strength              8.00 in      8.00 in
   Effective slab width            97.00 in
   Neg mom rebar area
          0.00 in2
   Rebar placement from bottom of slab
          4.00 in
   Negative mom. slab used in dead load 2 analysis
   Fillet                          2.50 in
   Effective slab width           97.00 in
   Self weight slab width        104.50 in

 Steel

   Web splice section  1
         Steel grade            M270-50W
   Web splice section  2
         Steel grade            M270-50W
   Web splice section  3
         Steel grade            M270-50W
   Rebar yield                  60.00 ksi

Output

   Standard resolution summary tables

Units

    Input units: U.S. cust.
   Output units: U.S. cust.
```

Girder I

PROJECT: Bent 2 (52-53' P/S-I)

ISOLATED FOOTING DESIGN

Code: AASHTO LRFD (3rd Edition) with 2005 Interims
Units: US

Geometry:
=========

Name : PileFoot1
Shape : Rectangular, Type : Pile/Shaft Cap

Bf(X) = 13.50 ft, Hf(Z) = 13.50 ft, Thickness(Y) = 54.00 in

Footing concentric.
Columns located on the footing:
Column No. 1 at x = 0.00 ft, Round D = 42.00 in

Ag = 182.25 ft^2, Ix = 150.00 ft^2, Iz = 150.00 ft^2

Surcharge = 0.72 ksf

Piles: Circular Size: 14.00 in Capacity: 120.00 kips

Pile Pattern Name: 13 x 13 (9 piles)
Pile Pattern concentric.
Pile Pattern Type: General
Number of Piles: 9

Design Parameters:
==================

f'c = 3000.00 psi fy = 60000.00 psi
phi flex = 0.90 phi shear = 0.90
Ec = 3156.0 ksi Es = 29000.0 ksi
Crack check as per 2005 Interims
Crack control Exposure = 1.00
Concrete Type : Normal Weight.

This Load controls over seismic

Pile Reactions, Service:
========================

Pile	Loc(X) ft	X in	Z in	comb	Ovs	P, kips	Mxx, kft	Mzz, kft	Pile Reac. kips
1	-5.05	21.0	-60.0	824	1.000	-919.51	-571.30	230.30	155.60*
				889	1.000	-497.37	571.30	-291.34	53.93
2	-0.05	81.0	-60.0	798	1.000	-891.43	-795.38	-50.18	153.81*
				852	1.000	-522.58	795.38	9.24	59.80
3	4.95	141.0	-60.0	798	1.000	-891.43	-795.38	-50.18	156.97*
				871	1.000	-527.25	795.38	30.07	60.20
4	-5.05	21.0	0.0	824	1.000	-919.51	-571.30	230.30	136.56*
				889	1.000	-497.37	571.30	-291.34	72.97
5	-0.05	81.0	0.0	824	1.000	-919.51	-571.30	230.30	130.42*
				878	1.000	-494.51	571.30	-271.24	83.19
6	4.95	141.0	0.0	668	1.000	-870.60	571.30	-258.79	135.06*
				1053	1.000	-548.09	-571.30	238.68	82.10
7	-5.05	21.0	60.0	642	1.000	-898.67	795.38	21.70	153.84*
				1019	1.000	-518.21	-795.38	-82.73	55.69
8	-0.05	81.0	60.0	642	1.000	-898.67	795.38	21.70	154.61*
				1008	1.000	-515.34	-795.38	-62.63	59.00

$\frac{156.97}{120} = 1.31$

seismic $= \frac{192.67}{180} = 1.0$

(plastic M about X + Z

PROJECT: Bent 2 (52-53' P/S-I)

9	4.95	141.0	60.0	642	1.000	-898.67	795.38	21.70	155.39*
				1027	1.000	-520.01	-795.38	-41.80	61.77

Pile Reactions, Factored:

Pile	Loc(X) ft	X in	Z in	Column Loads comb	Ovs	P, kips	Mxx, kft	Mzz, kft	Pile Reac. kips
1	-5.05	21.0	-60.0	27	---	-1294.90	-880.34	119.49	211.82
				170	---	-450.41	880.34	-201.43	50.00
2	-0.05	81.0	-60.0	27	---	-1294.90	-880.34	119.49	209.99
				159	---	-445.39	880.34	-166.27	56.91
3	4.95	141.0	-60.0	27	---	-1294.90	-880.34	119.49	208.17
				221	---	-507.68	660.12	300.17	62.01
4	-5.05	21.0	0.0	27	---	-1294.90	-880.34	119.49	182.47
				222	---	-434.35	27.28	-436.09	69.77
5	-0.05	81.0	0.0	27	---	-1294.90	-880.34	119.49	180.65
				222	---	-434.35	27.28	-436.09	85.03
6	4.95	141.0	0.0	1	---	-1267.80	880.34	-149.45	184.73
				224	---	-517.33	-27.28	404.47	81.63
7	-5.05	21.0	60.0	1	---	-1267.80	880.34	-149.45	199.89
				223	---	-444.01	-660.12	-331.78	52.30
8	-0.05	81.0	60.0	1	---	-1267.80	880.34	-149.45	206.98
				185	---	-472.49	-880.34	102.67	59.92
9	4.95	141.0	60.0	1	---	-1267.80	880.34	-149.45	214.07
				204	---	-480.65	-880.34	139.12	56.99

Note:
* Force in pile is greater than pile capacity.

Only max. force in piles is considered for design.
Pile coordinates X and Z are from the most left edge of the footing.

Max. Pile Reaction Used in Design: (without selfweight and surcharge)

Factored pile reaction = 177.31 kips

Service pile reaction = 128.72 kips

Fatigue pile reaction = 47.45 kips

Reinforcement Schedule:

Dir	Quantity	Size	Bar dist. in	As total in^2	Spacing in	Hook
X	15	# 8	4.50	11.85	10.93	None
X	9	# 8	49.50	7.11	19.13	None
Z	15	# 8	5.50	11.85	10.93	None
Z	9	# 8	48.50	7.11	19.13	None

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Flexure:
========

Dir	Loc ft	d in	Mmax kft	Comb	Asb_req in^2	Asb_prv in^2	Asb_eff in^2	Ast_req in^2	Ast_prv in^2	Ast_eff in^2
X	-1.55	49.50	1861.2	1	11.30	11.85	11.85	6.56	7.11	7.11
X	1.55	49.50	1808.0	1	10.97	11.85	11.85	6.56	7.11	7.11
Z	-1.55	48.50	1834.6	1	11.37	11.85	11.85	6.56	7.11	7.11
Z	1.55	48.50	1834.6	1	11.37	11.85	11.85	6.56	7.11	7.11

Cracking/Fatigue
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Cracking check as per AASHTO LRFD 3rd Edition with Interims (2005)

			<-------- Cracking	----------	>			<---------- Fatigue	---------	>	
Dir	Loc ft	d in	Mmax kft	Comb	fs ksi	Srq in	Spr in	Mmax kft	Comb	fs ksi	ratio fs
X	-1.55	49.50	1351.2	798	29.16	12.2	10.6	498.1	1059	10.75	0.57
X	1.55	49.50	1312.6	798	28.32	12.9	10.6	483.8	1059	10.44	0.55
Z	-1.55	48.50	1331.9	798	29.37	9.5	10.6*	491.0	1059	10.82	0.57
Z	1.55	48.50	1331.9	798	29.37	9.5	10.6*	491.0	1059	10.82	0.57

Note:
* Provided rebar spacing is not adequate for crack control.

One Way Shear:
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(Simplified Method)

Col	Dir	Dist ft	Comb	dv in	Vu kips	phi*Vc kips
1	X	-5.65	1	49.15	0.0	785.0
	X	5.65	1	49.18	0.0	785.5
	Z	-5.56	1	48.14	0.0	768.9
	Z	5.56	1	48.14	0.0	768.9

Two Way Shear:
==============

#	Bo ft	Ao ft^2	Comb	Avg. dv in	Vu kips	phi*Vc kips
Columns:						
1	23.73	44.82	1	48.65	1418.4	2721.7
Piles - max:						
5	16.40	21.41	1	48.65	177.3	1881.0
Piles - min:						
1	7.60	17.55	1	48.65	177.3	871.6

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Note:
TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.