# AASHTO T-3 TRIAL DESIGN BRIDGE DESCRIPTION

State: *Missouri*

Trial Design Designation: *MO-2*

Bridge Name: *Missouri Bridge*

Superstructure Type: *Prestressed precast concrete "I" girder*

Span Length(s): *Three spans @ 59.2ft.-60.0ft.-59.2ft.*

Substructure Type: *Three 3.0ft dia. reinforced concrete columns per bent*

Foundation: *Cast-in-place concrete pile with 14in. dia. steel casing*

Abutments: *Integral diaphragm wall supported on 14in. dia. CIP pile*

Seismic Design Category (SDC): *"D"*

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

Design Spectral Acceleration at 1-second Period ($S_{D1}$): *0.86g*

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

# TABLE OF CONTENTS



**Appendix**

Purpose:

Review "Recommended LRFD Guidelines for the Seismic design of highway bridges". For review considered P/S I Girder (60'-60'-60') span bridge with zero degree skew. Roadway width is 38'-10"; Int. bents with 3 columns on foundation (CIP) & Abutment bents are Integral. This bridge was designed as per AASHTO (10% probability of exceedance in 50 years, Approximately 500 years return period).

Assumptions/comments:

1. Bridge location: 36 deg Lat. & -89.817 deg Long.

2. Seismic category: SDC "D".

3. Use web site: http://earthquake.usgs.gov/research/hazmaps (2002 USGS data updated 2003) & generate acceleration values for 5% PE in 50 years at 1.o Hz (1.o sec, **S1**) & 5.0 Hz (0.2 sec, **Ss**) using Latitude & Longitude values. (See sheet no. 14 for Response Spectrum curve).

   Revised (reduced acceleration) values for 1000 years period (Maps & Data) were available in June 2006. (See sheet no. 12A for Response Spectrum curve).

   Since seismic analysis was finished with higher acceleration values (2002 USGS data), seismic analysis is not revised for reduced acceleration values (June 2006 USGS data & maps).

4. Response spectrum created considering earthquake ground motions for the 5% probability of exceedance in 50 years (Approximately 1000 years return period) using 2002 USGS data.

5. As per LRFD guideline section 8.4.4, expected concrete compressive strength f'ce shall be the greater of 1.3*f'c or 5000 psi. But in the XTRACT program used smaller of 1.3*f'c or 5000 psi to achieve failing material from steel to concrete.

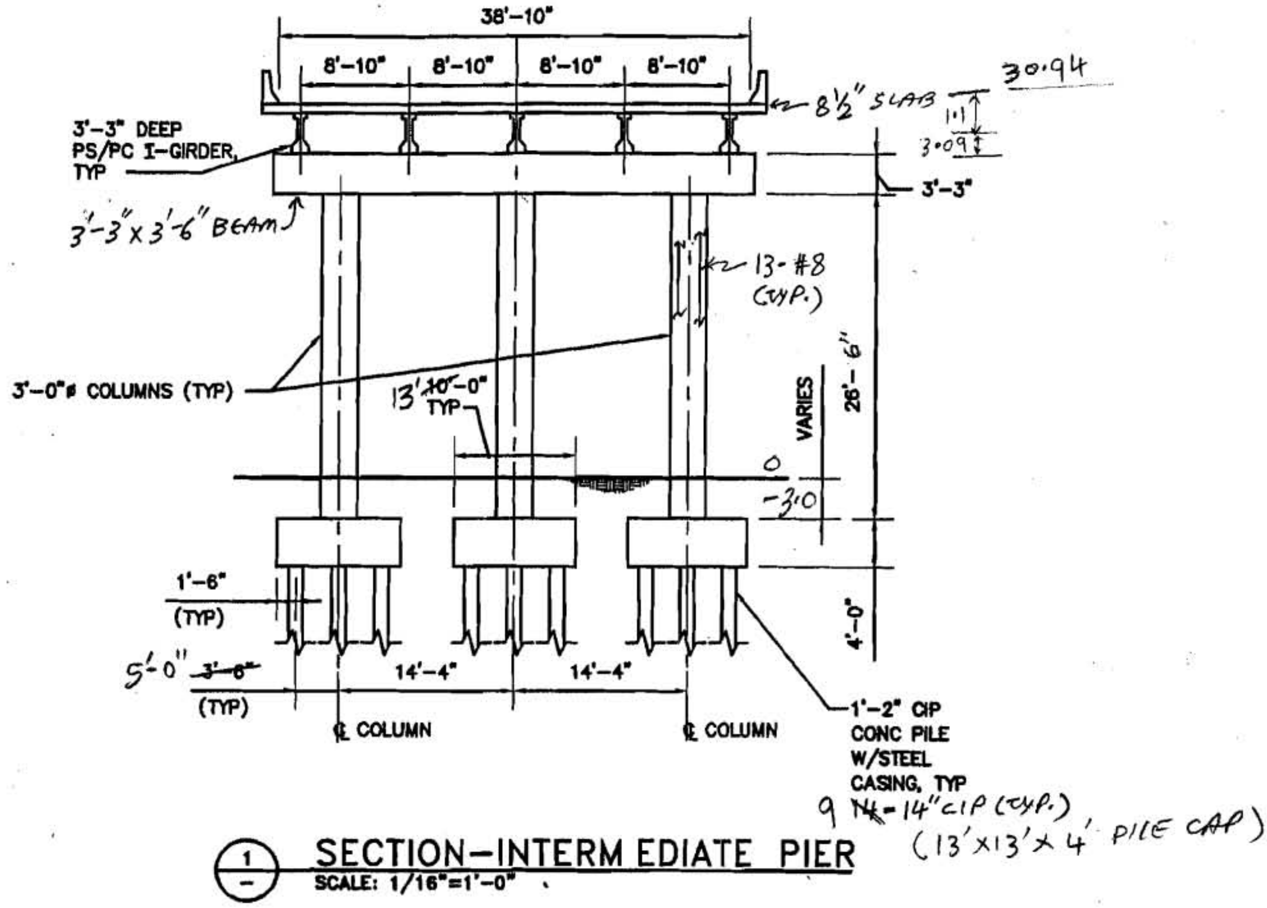
6. Assumed all seismic loads will be resisted by Int. bents only (without any contribution from Abutment/End bents).

7. Int. bents are checked only for Trans. Direction force using XTRACT program & CAPP program (Pushover analysis).

8. Assumed pinned at bottom end of column & fixed at top end of column.

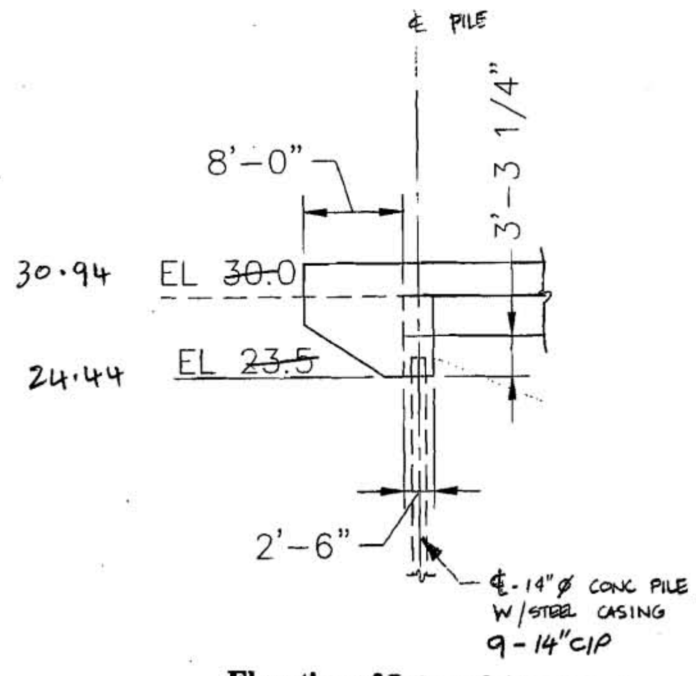
9. Liquefaction is not considered.

EXAMPLE 1 (MO) SP 

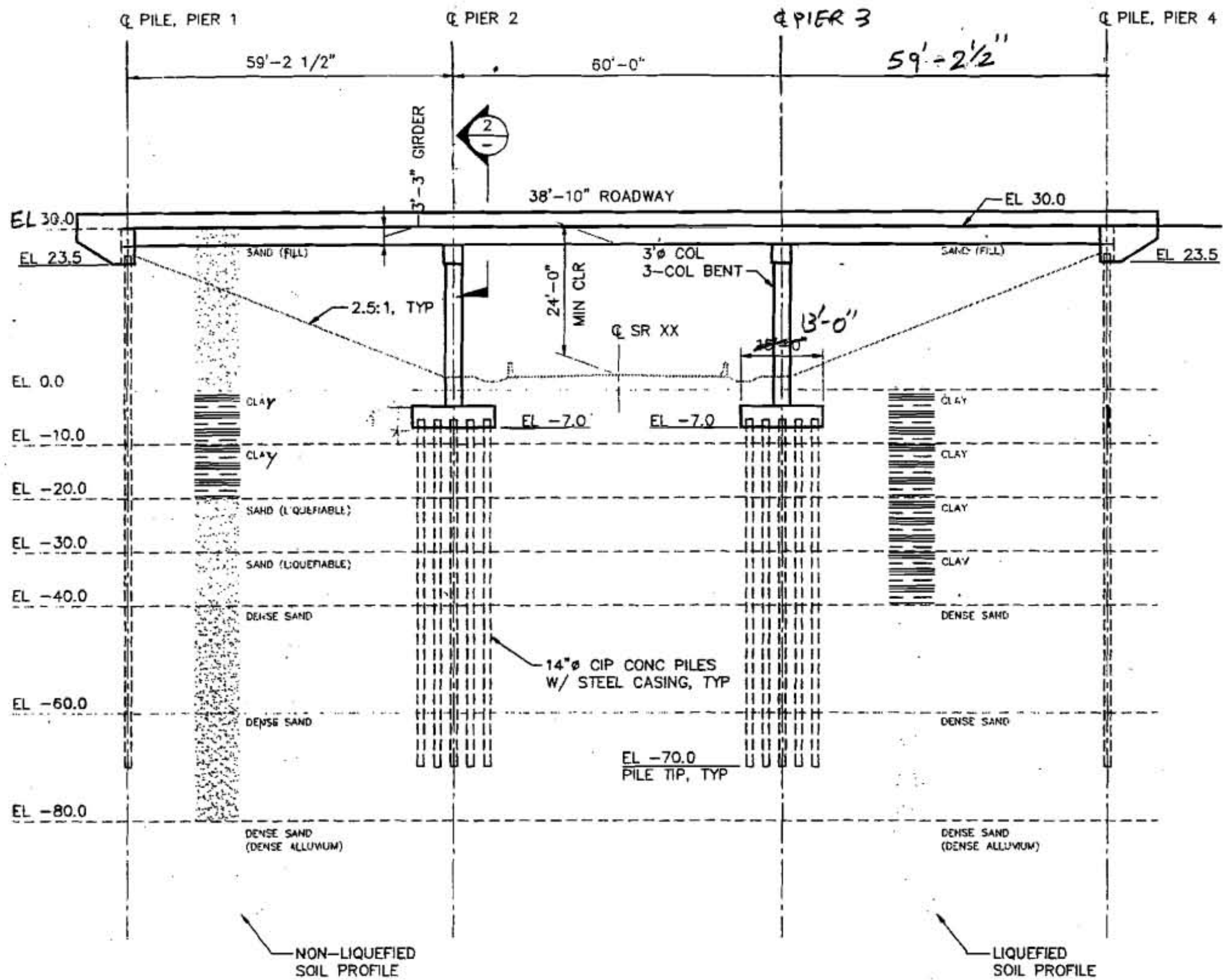



SECTION–INTERMEDIATE PIER
SCALE: 1/16"=1'-0"

**Elevation of Intermediate Pier**




**Elevation of Integral Abutment**




EXAMPLE 1 (MO)

SP

3

1

ELEVATION – MISSOURI BRIDGE

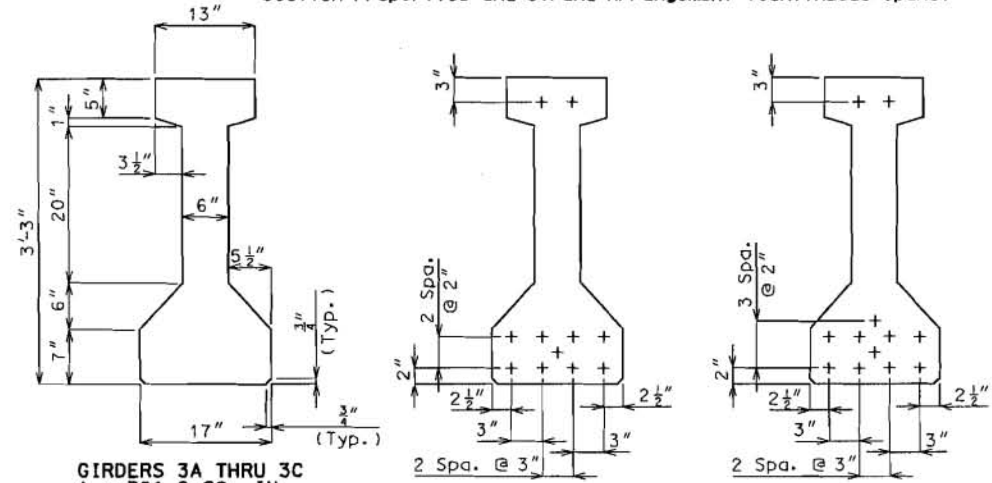
SCALE: 1" = 15'-0"

Example1 (MO) SP 4



3.3 Beam Type 3

Section Properties and Strand Arrangement (Continuous Spans)




GIRDERS 3A THRU 3C
A = 381.9 SQ. IN.
$Y_b$ = 17.08 IN.
I = 61,841 IN.$^4$

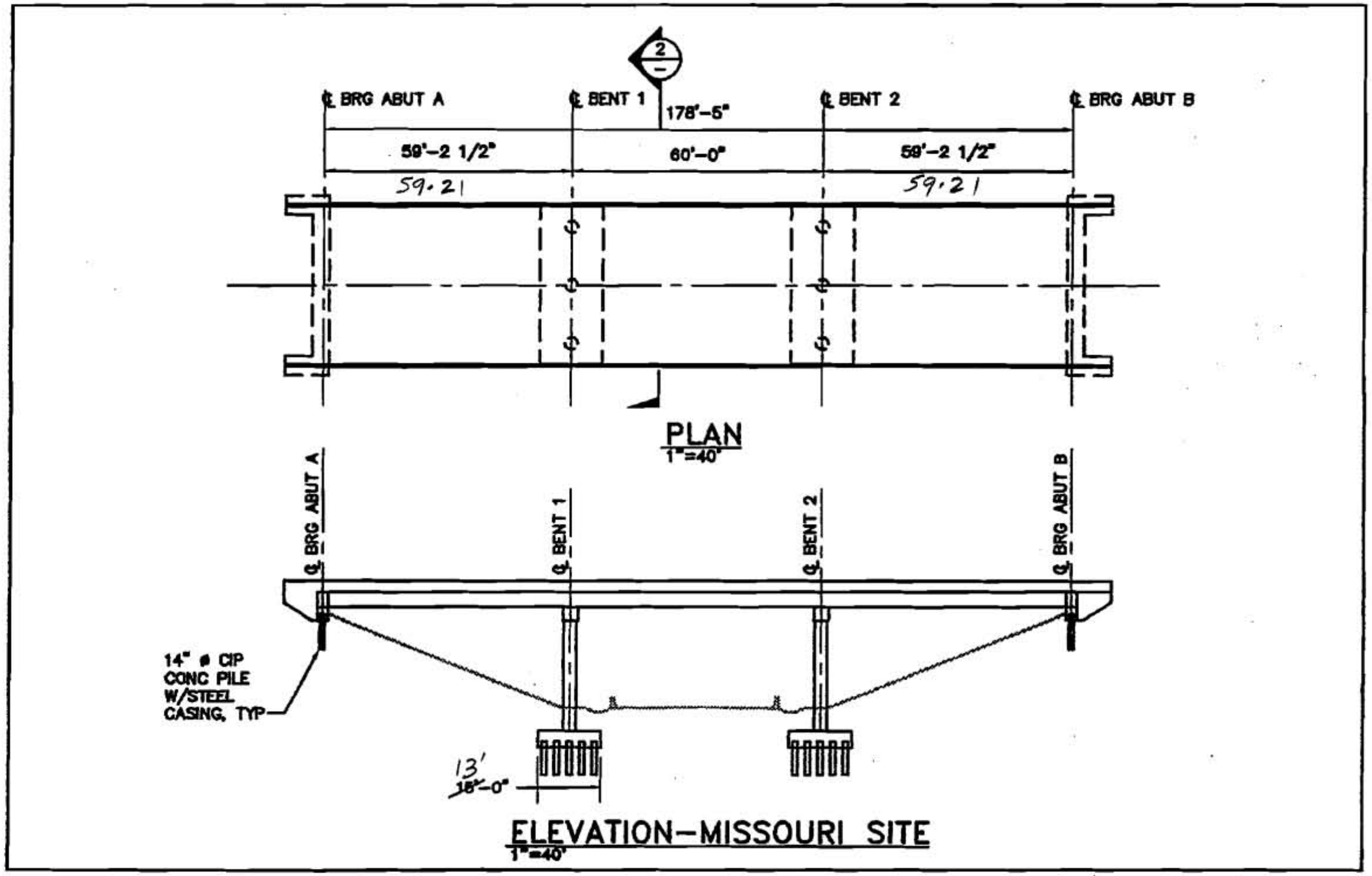
GIRDER 3A
(11 STRANDS)

GIRDER 3B
(12 STRANDS)

| GIRDER SEQ. NO. | | 3A | 3B |
|---|---|---|---|
| Initial Prestress | kips | 341 | 372 |
| Size of Strands | in. | 1/2 | 1/2 |
| No. of Strands (All Straight) | | 11 | 12 |
| Bottom of Girder to Center of Gravity of Strands | inches | 9.82 | 9.67 |

NOTE: Investigate the possibility of using all straight strands when strength check of a hold-down device exceeds allowable.
<u>All strand arrangements shown on this page have straight strands only.</u>

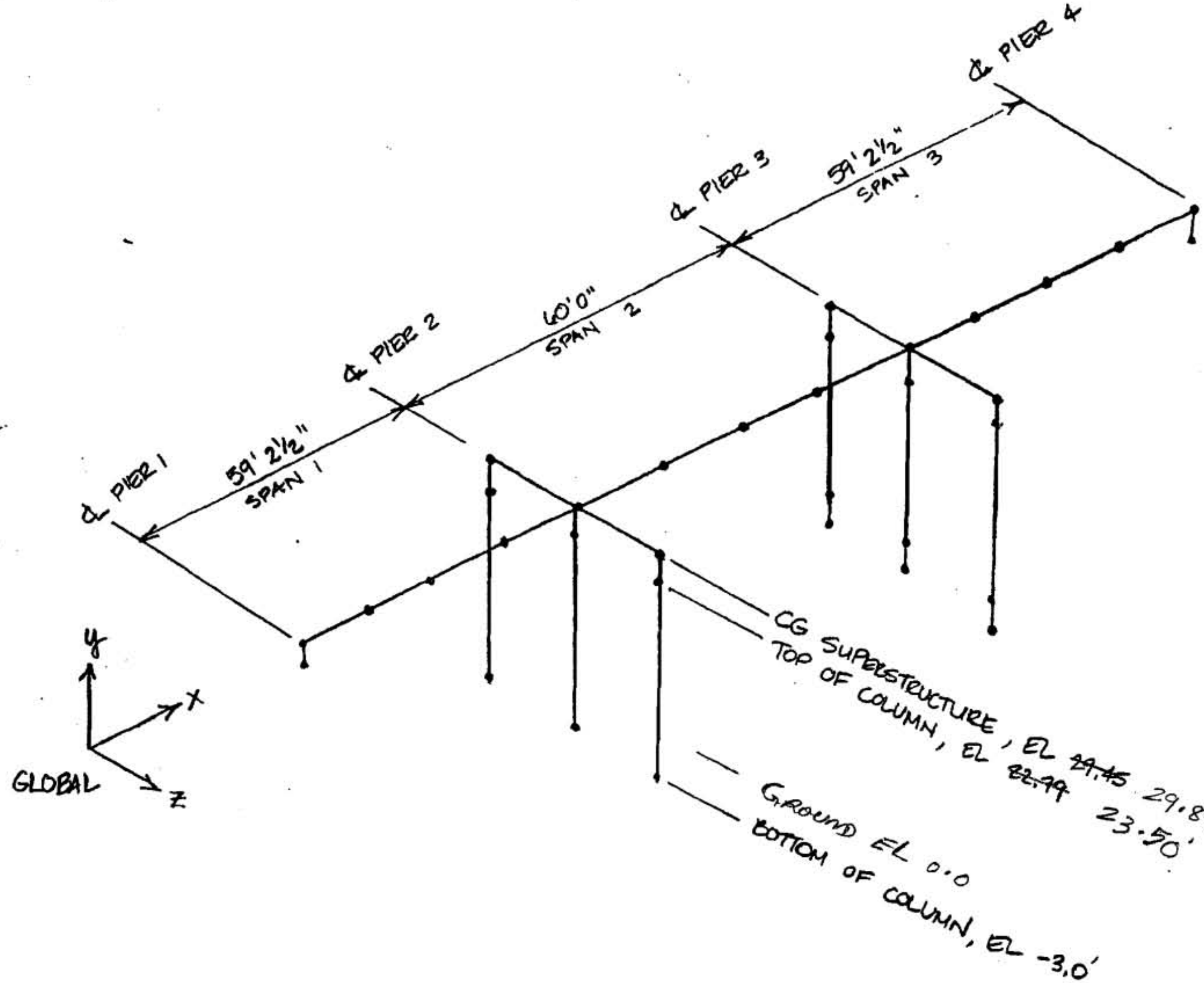
Strand arrangements other than those shown may be investigated by the designer.




EXAMPLE 1 (MO)

SP

5




EXAMPLE 1 (MO)

8

6

**SEISMIC DESIGN**

**Acceleration coefficient = A =**
**Performance category =**
**soil type =**
**slab thickness =** 8.50 inch
Span no. 1.00
span length = 60.00 ft
Roadway width = 38.83 ft
Assume Haunch = 1.75 inch
Fc' of slab = 4000.00 ksi
Fc' of Grd = 5000.00 ksi
Grd Ytop = 21.92 inch
**Grd Ht. =** 39.00 inch
Grd Ybot = 17.08 inch
Ratio of Fc' = 1.12
Slab Area = 27.51

Girder infomation :

| element | width | thickness |
|---|---|---|
| top flg | 1.06 | 0.50 |
| web | 0.50 | 1.92 |
| bot flg | 1.42 | 0.83 |
| no of girder | 5.00 | |

Grd area = 381.90 in^2 2.65 Ft^2
Iz grd = 61.84 in^4 0.0030 Ft^4

0.20

| grd to grd | bay1 | bay2 | bay3 | bay4 | bay5 | bay6 | bay7 | bay8 | bay9 |
|---|---|---|---|---|---|---|---|---|---|
| 35.33 | 8.83 | 8.83 | 8.83 | 8.83 | | | | | |

y for haunch 0.78
**C.G. =** 0.35 ft from top of slab
Iy' @ c.g. = 3456.77
Iz' @ c.g. = 1.15
y for grd = 2.68

Haunch
Iy' @ c.g. = 0.02
Iz' @ c.g. = 0.00
Area = 0.16

**MOMENT OF INERTIA OF SUPERSTRUCTURE**

| NO. | AREA FT2 | Z FT | AREA * Z FT3 | IND. INT.,Iy' FT4 | A*dZ2 FT4 | Iy(TRANSF) TOTAL INT. FT4 | Y FT | AREA * Y FT3 | Iz' IND. INT., FT4 | A*dY2 FT4 | Iz(TRANSFO) TOTAL INT. FT4 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SLAB | **27.51** | 0.00 | 0.00 | **3456.77** | 0.00 | 3456.77 | **0.35** | 9.74 | **1.1501** | 15.49 | 16.64 |
| GD1 | **2.65** | -17.67 | -46.85 | **0.001** | 827.74 | 827.75 | **2.68** | 7.11 | **0.0030** | 6.59 | 6.59 |
| GD2 | **2.65** | -8.83 | -23.43 | **0.001** | 206.94 | 206.94 | **2.68** | 7.11 | **0.0030** | 6.59 | 6.59 |
| GD3 | **2.65** | 0.00 | 0.00 | **0.001** | 0.00 | 0.00 | **2.68** | 7.11 | **0.0030** | 6.59 | 6.59 |
| GD4 | **2.65** | 8.83 | 23.43 | **0.001** | 206.94 | 206.94 | **2.68** | 7.11 | **0.0030** | 6.59 | 6.59 |
| GD5 | **2.65** | **17.67** | 46.85 | **0.001** | 827.74 | 827.75 | **2.68** | 7.11 | **0.0030** | 6.59 | 6.59 |
| GD6 | | | | | | | | | | | |
| GD7 | | | | | | | | | | | |
| GD8 | | | | | | | | | | | |
| GD9 | | | | | | | | | | | |
| GD10 | | | | | | | | | | | |
| HAUNCH1 | **0.16** | -17.67 | -2.79 | **0.02** | 49.31 | 49.32 | **0.78** | 0.12 | 0.0003 | 0.02 | 0.02 |
| HAUNCH2 | **0.16** | -8.83 | -1.40 | **0.02** | 12.33 | 12.34 | **0.78** | 0.12 | **0.00** | 0.02 | 0.02 |
| HAUNCH3 | **0.16** | 0.00 | 0.00 | **0.02** | 0.00 | 0.02 | **0.78** | 0.12 | **0.00** | 0.02 | 0.02 |
| HAUNCH4 | **0.16** | 8.83 | 1.40 | **0.02** | 12.33 | 12.34 | **0.78** | 0.12 | **0.00** | 0.02 | 0.02 |
| HAUNCH5 | **0.16** | 17.67 | 2.79 | **0.02** | 49.31 | 49.32 | **0.78** | 0.12 | **0.00** | 0.02 | 0.02 |
| HAUNCH6 | | | | | | | | | | | |
| HAUNCH7 | | | | | | | | | | | |
| HAUNCH8 | | | | | | | | | | | |
| HAUNCH9 | | | | | | | | | | | |
| HAUNCH10 | | | | | | | | | | | |
| TOTAL= | 41.56 FT2 | | 0.00 | | | **5649.49** FT4 | | 45.91 FT3 | | | **49.69** |

ZB= **0.00** FT FROM CENT OF STRUCTURE Yb = **1.10** FT FROM TOP OF SLAB

I 22 = Iyy= **5649.49** FT4 I33 = Izz = **49.69** FT4

**Torsional moment of inertia, Ix = I11 :**

roadway 38.83
slab thick 0.71
haunch 0.15

4.8025

I11 slab = 4.55

I11 Grd = 2.23

I11 Haunch 0.01

**Total I 11 = I xx = 6.78**

| | | |
|---|---|---|
| Total Area = | **41.56** | FT2 |
| I 11 = | **6.78** | FT4 |
| I 22 = | **5649.49** | FT4 |
| I 33 = | **49.69** | FT4 |

**Superstruct Dead load**

| | | | |
|---|---|---|---|
| Fws | 35.00 | psf | |
| Slab thick | 8.50 | inch | |
| Roadway | 38.83 | ft | |
| No.of girder | 5.00 | | |
| Girder area | 2.65 | | |
| Girder spac | 8.83 | | |
| Grd depth | 3.25 | ft | |
| Grd bf bottom | 17.00 | inch | |
| Edge from Grd | 6.00 | inch | |
| Barrier curb area | 2.28 | | |
| No. of curb | 2 | | |
| **Int. bent no.** | **2** | **3** | |
| No. of column | 3 | 3 | |
| **Int. diaph** | | | |
| Brg ht. assumed | 1.00 | 1.00 | inch |
| Grd embed | 8.25 | 8.25 | inch |
| Diap length | 37.25 | 37.25 | ft |
| Diap width | 2.50 | 2.50 | ft |
| Lt span | 59.21 | 60.00 | ft |
| Rt span | 60.00 | 59.21 | ft |
| Avg span | 59.60 | 59.61 | ft |
| col size | 3 | 3 | ft |
| col length | 26.50 | 26.50 | ft |
| **col wt.** | 28.10 | 28.10 | kips/col |
| Beam width | 3.50 | 3.50 | ft |
| Beam Depth | 3.25 | 3.25 | ft |
| Beam length | 38.25 | 38.25 | ft |
| **Beam wt** | 65.26 | 65.26 | kips |
| Int. diaph wt | 43.78 | 43.78 | kips |
| Beam wt. + Int. diaph wt = | **109** | **109** | kips |
| Super struct DL | 486.26 | 486.27 | kips |
| Total DL @ bent = Super struct DL + Int diaph wt= | 530.04 | 530.05 | kips |
| **DL per column w/o substruct** | **177** | **177** | kips |
| **DL per column with substruct** | **227** | **227** | kips |

w/o substructure

w/o Footing

Recommended LRFD Guidelines NCHRP 20-7(193)

| | | |
|---|---|---|
| Fws | 1.36 | k/ft |
| Slab | 4.13 | k/ft |
| Girders | 1.99 | k/ft |
| Curbs | 0.68 | k/ft |

Supaerstr DL = W = **8.16** k/ft

**Check column requirement**

**SDC** D

| | | | | |
|---|---|---|---|---|
| **Art. 8.7.1** | **Min Lateral strength** | | | |
| | **Bent no.** | 2 | 3 | |
| | Col size = | 3 | 3 | ft |
| | Col Rebar size = | #8 | #8 | |
| | No. of bar = | 13 | 13 | |
| | Col bar As = | 10.21 | 10.21 | in^2 |
| Article 8.8.2 | Min %steel req'd = | 1.00 | 1.00 | |
| | % steel provided = | **1.00** | **1.00** | <= 4% |
| Article 8.8.1 | Max 4%steel allow = | O.K. | O.K. | |
| | 0.1 P DL = | 17.67 | 17.67 | kips |
| | Dist from C.G. of superstr to top of footing = | 32.84 | 32.84 | ft |
| | Moment = | 580 | 580 | k-ft |
| | Axial force = | 177 | 177 | kips |
| | Mom capacity from BM LRFD 3.71 2.5-2 = | 650 | 650 | k-ft |
| | | O.K. | O.K. | |
| **Art. 8.7.2** | **Max Axial Load** | | | |
| | Ag = | 7.07 | 7.07 | Ft^2 |
| | F'ce = | 3.90 | 3.90 | ksi |
| | 0.2F'ce*Ag | 794 | 794 | kips |
| | Axial Load = | 400 | 400 | kips |
| | | O.K. | O.K. | |

## Use Cracked Mom Inertia in SEISAB run

| | | | | | |
|---|---|---|---|---|---|
| | | Int. Bent No. | 2 | 3 | |
| | | cover for Hoop = | 1.5 | 1.5 | in |
| | | Column Dia. = | 3 | 3 | ft. |
| | | Assume Hoop bar size = | 5 | 5 | |
| | | Bar diam., db = | 0.625 | 0.625 | in |
| | | Hoop bar area, As = | 0.31 | 0.31 | sq. in |
| | | Hoop Spacing, S = | 3 | 3 | in |
| | | Hoop diam., Dc = | 33 | 33 | in |
| | | Fc = | 3.00 | 3.00 | ksi |
| | | Col. Long. Reinf. Ast = | **10.21** | **10.21** | sq. in |
| | | Column Gross Area, Ag = | 1018 | 1018 | sq. in |
| | | DL max per column from superstruct | **177.00** | **177.00** | k |
| | | Assumed EQ load | **0.00** | **0.00** | k |
| Eq. 8.7 | factor | **1.3** F'ce = 1.3*Fc' | **3.90** | **3.90** | ksi |
| | | **P/(F'ce*Ag)** | **0.04** | **0.04** | |
| | | **Ratio Ast/Ag =** | **1.00** | **1.00** | % |
| 254.469 | | From chart Ie/Ig Ratio | **0.40** | **0.40** | **NCHRP 20-7 page 5-18** |
| | | Ig = Pi*R^4/4 | **3.98** | **3.98** | **Gorss Mom Int. (Ft^4)** |
| | | I 22 = Ratio*Ig = | **1.59** | **1.59** | **Crack Mom Int. (Ft^4)** |
| | | I 33 = Ratio*Ig = | **1.59** | **1.59** | **Crack Mom Int. (Ft^4)** |
| | | Torsional Mom intertia, Jg = pi*R^4/2 | **7.95** | **7.95** | **Gorss Torsional Mom Int. (Ft^4)** |
| Eq. 5.6 | factor | **0.2** J eff = I 11 = Factor*Jg | **1.59** | **1.59** | **Effective Torsional Mom int.** |
| | | Column Gross Area, Ag = | **7.07** | **7.07** | **Sq. Ft** |
| | | Mod Elastic, E = | **545187.1664** | **545187.1664** | **KSF** |

32979.18 in^4

| W, pcf | F'c, ksi | E, ksf | E, ksi |
|---|---|---|---|
| 150 | **3.0** | **478161** | **3321** |
| 150 | **4.0** | **552133** | **3834** |
| | **F'ce, ksi** | **Emod, ksf** | **Emod, ksi** |
| 150 | **3.9** | **545187** | **3786** |
| 150 | **5.0** | **617303** | **4287** |
| Light wt 140 | **3.0** | **431151** | **2994** |

Eqn 8.7

USE 3.9 ksi INSTEAD of 5.0 ksi OTHERWISE STEEL FAIL in XTRACT ANALYSIS

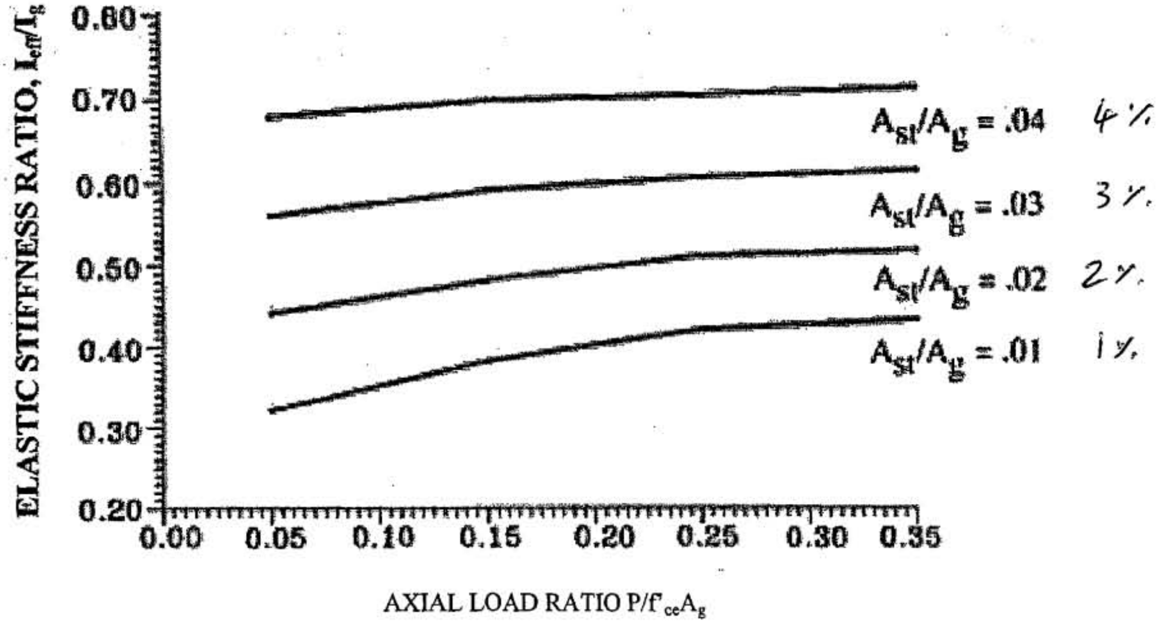
**Rebar properties**

| | Ksi | Ksf | | |
|---|---|---|---|---|
| **Esteel** | **29000** | 4176000 | | |
| **Yield strength, Fy =** | **60** | ksi | | |
| **Expected Yield strength, Fye = 1.1* Fy =** | **66** | ksi | Eq. 8.1 | |
| **Expected Tensile strength, Fue = 1.4* Fye =** | **92.4** | ksi | Eq. 8.2 | |

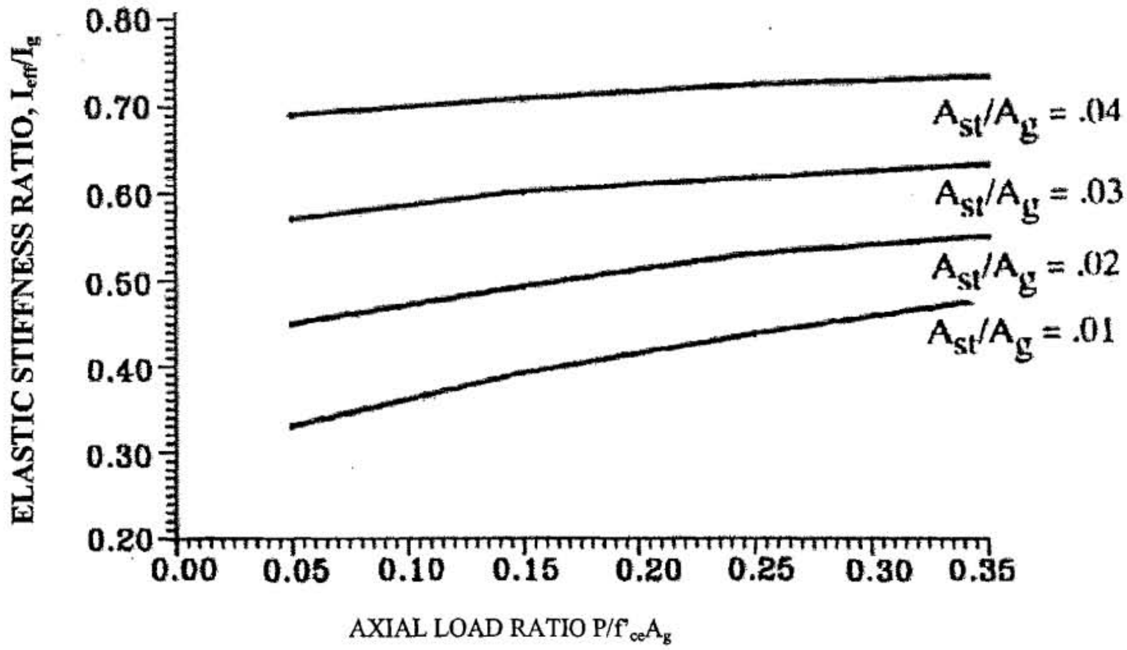
## BEAM Properties

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| | width, b | Ht. | | | No of bar | Assume bar size # | |
| **Beam Size** | 3.5 | 3.25 | Ft | | 9 | 9 | |
| Bar area, As = | 9 | in^2 | | Use F'ce= | **3.90** | ksi | |
| n = Es/Ec = | 7.66 | | | **Ec =** | **3786** | ksi | |
| r = As/bd | 0.005871 | | | d = | 36.5 | inch | |
| n*r = | 0.044969 | | | | | | |
| k = (nr^2+2nr)^1/2-nr | 0.25828092 | | | | | | |
| J = 1-k/3 = | 0.913906 | | | | | | |
| Special | **I crac =** bd^3*(1/2k^2*J) = | | | **3.00** | ft^4 | Beam area = | **11.38** ft^2 |
| | | | | 62256 | in^4 | | 1638 in^2 |

EXAMPLE 1 (MO) SP 10




a) Circular Sections




b) Rectangular Sections

**FIGURE 5.4** **Effective Flexural Stiffness of Cracked Reinforced Concrete Sections [x]**

**Interactive Deaggregations, 2002**

On this page you may select a return time, SA frequency, specify a latitude and longitude and request seismograms. Links to the following information will be returned:

- A plot of deaggregated distance, magnitude and ground-motion uncertainty for the specified parameters (gif, pdf, ps).
- An ascii text file of the hazard matrices, containing, but not limited to, the frequency selected.
- A geographic deaggregation plot may also be specified (for designated frequencies only - see below). This is in addition to the plot mentioned above.
- An ascii text file and graph of the seismograms for the modal or mean event (if requested).

README is a page containing information on how the deaggregation is done and about the input parameters to the program. It will increase your likelihood of success with this site if you read it first. Stochastic Seismograms and What is Epsilon? are articles which discuss the theory behind the seismograms.

On some browsers you have to click on a pre-selected item in a list to deselect it. If you select an item without doing this you will have two items on the list selected and you will get a broken icon instead of a plot!

**Site name:**

*Used for plot labeling purposes only*
underscore (_), comma (,) and alphanumeric characters only,
***no blanks (they will be replaced with an underscore)***,
name length <= 16 characters.

Name: Enter site name

**Select location of interest in latitude/longitude:**

Specify in decimal degrees, use "-" to specify western longitudes.
**Conterminous US:** latitude 25 to 49 degrees, longitude -125 to -65 degrees, only.
**Alaska:** refer to 1996 Interactive Deaggregations page.
**Hawaii:** refer to 1996 Interactive Deaggregations page.
**Puerto Rico:** latitude 17 to 19 degrees, longitude -64 to -68 degrees, only.

Latitude: Enter lat Longitude: Enter lon

-ive value

**Return time:**

PE = probability of exceedance
Select one!

- 1% PE in 50 years
- 2% PE in 50 years
- 5% PE in 50 years
- 10% PE in 50 yrs

**SA frequency:**

SA = Spectral Acceleration;
PGA = peak ground acceleration.
**Puerto Rico:** only 0.5 hz, 1.0 hz, 5.0 hz and PGA are available

- 0.5 hz
- 1.0 hz
- 2.0 hz
- 3.33 hz

1.0 HZ FOR 1 SEC
5.0 HZ FOR 0.2 SEC.

**Geographic Deaggregation:**

Make a map with hazard bars at source locations.

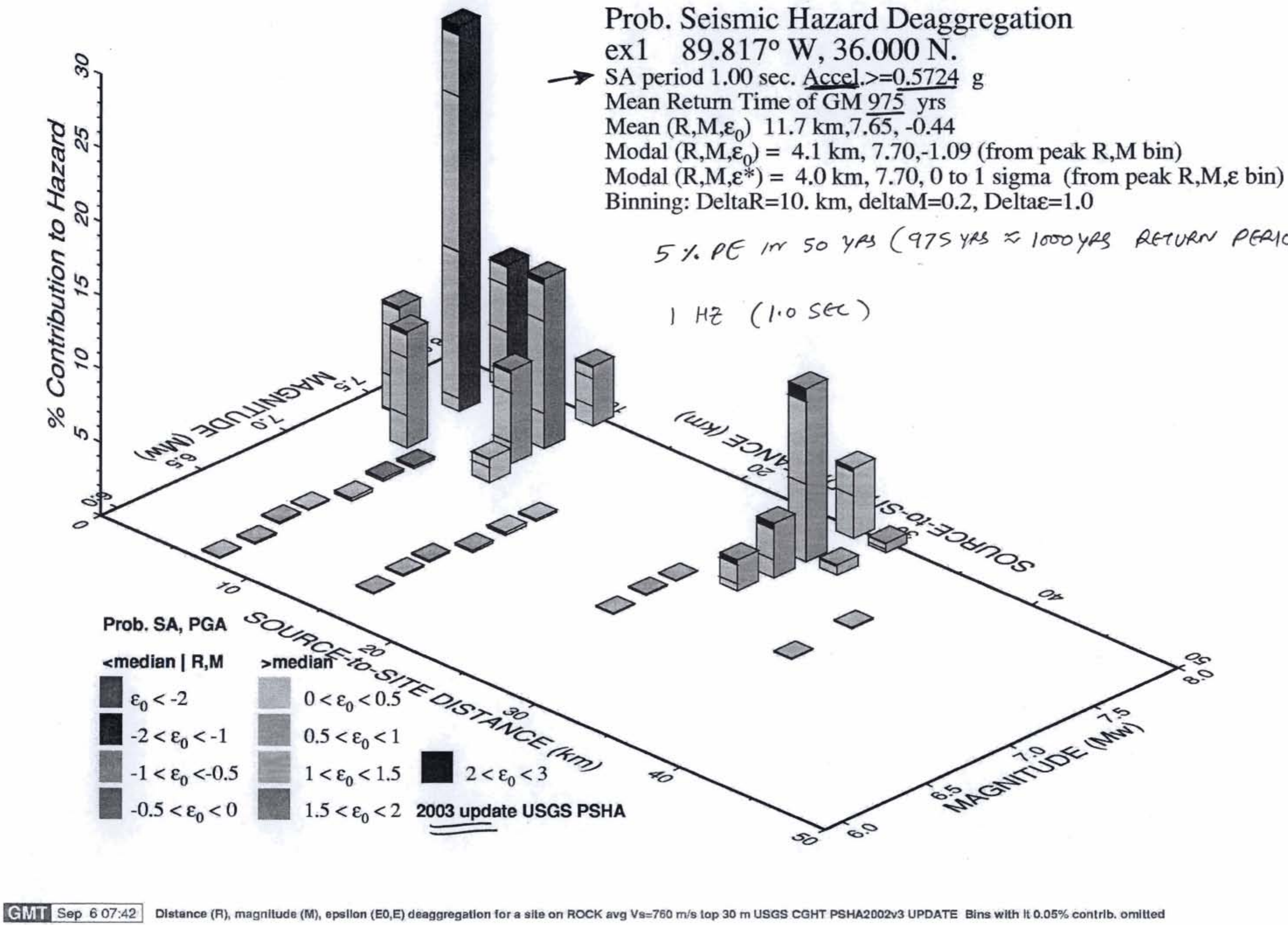
- (•) None
- ( ) Coarse angle, coarse distance
- ( ) Fine angle, coarse distance
- ( ) Coarse angle, fine distance
- ( ) Fine angle, fine distance

**Seismograms:**

Do you want seismograms for the Modal or Mean event?

- (•) None
- ( ) Modal, one-corner source
- ( ) Mean, one-corner source

**It may take several minutes to generate the plot(s) and do file conversions**

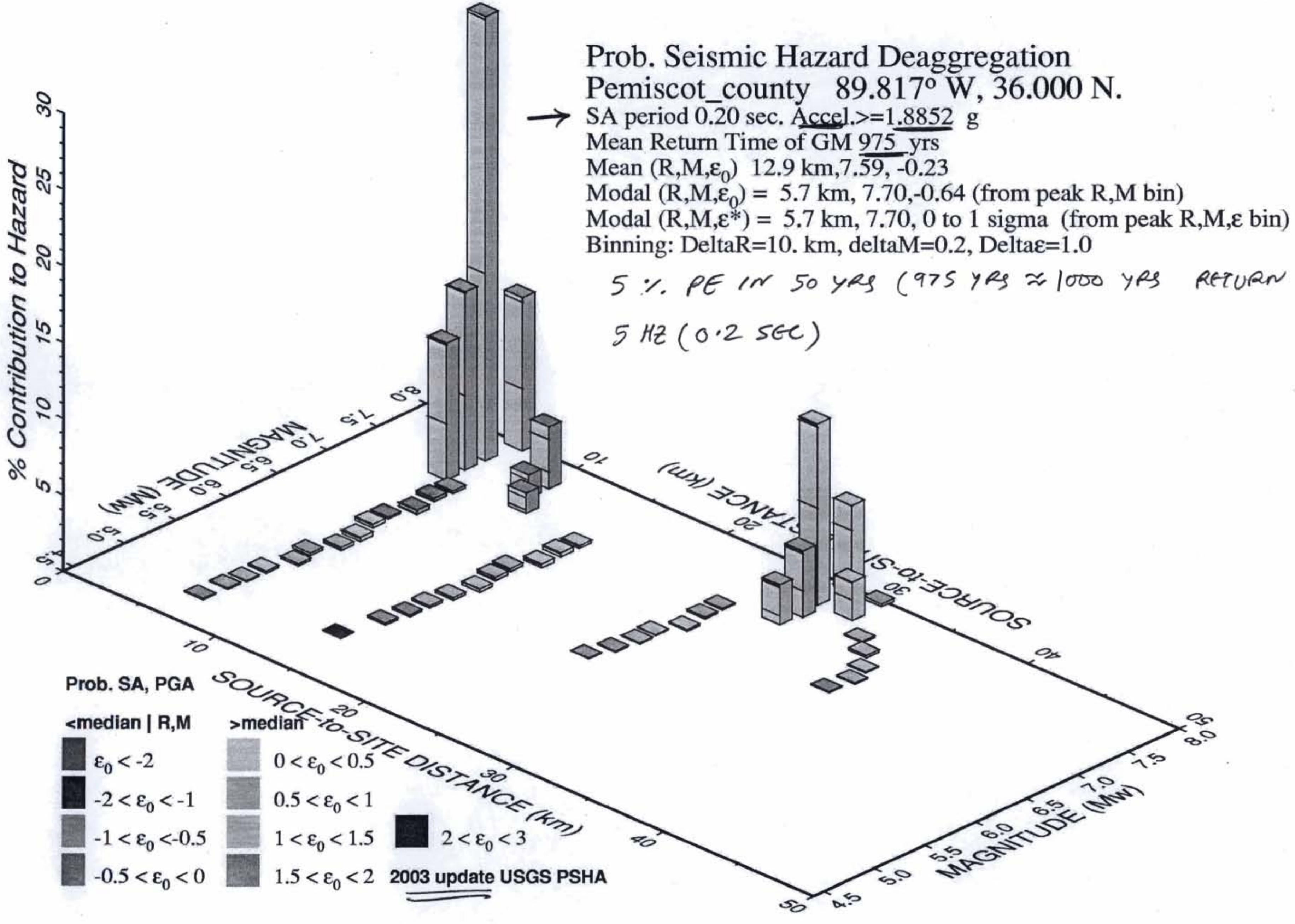



5% PE in 50 yrs (975 yrs ≈ 1000 yrs return period)

1 Hz (1.0 sec)

EXAMPLE1 (MO)

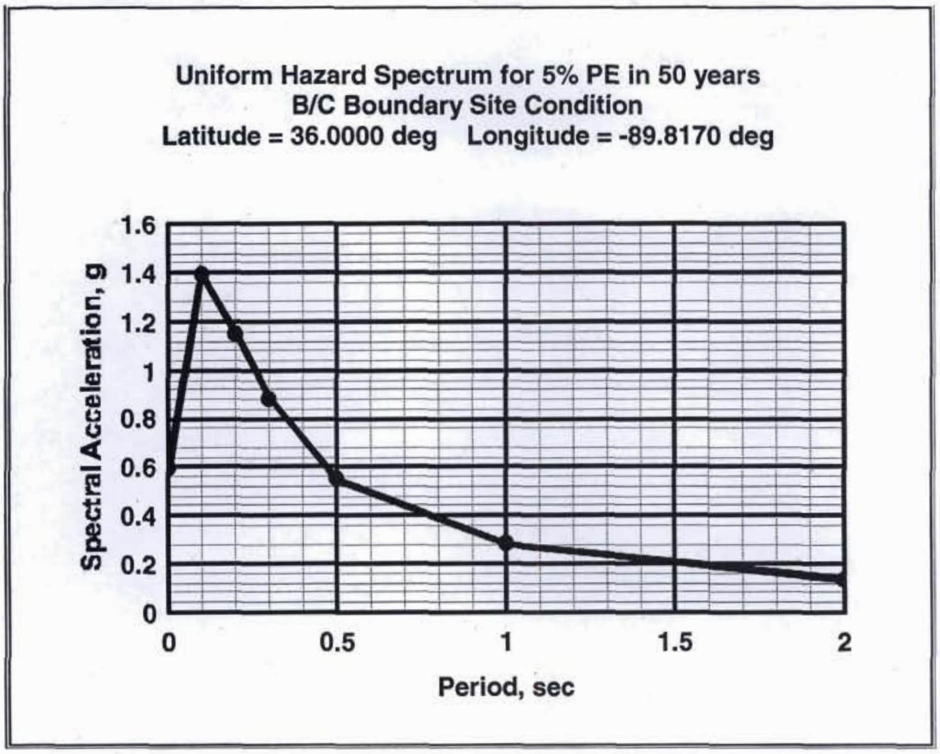
SP




EXAMPLE 1 (Mo)

SP

12




| Period, sec | Sa, g |
|---|---|
| 0.0 | 0.597 |
| 0.1 | 1.395 |
| 0.2 | 1.149 |
| 0.3 | 0.885 |
| 0.5 | 0.549 |
| 1.0 | 0.282 |
| 2.0 | 0.135 |

NOTE: INFORMATION AS PER LATEST AVAILABLE USGS DATA & MAP FOR 1000 YR PERIOD.

| | AS PER LATEST DATA ** | AS PER 2002 USGS DATA |
|---|---|---|
| 0.2 SEC | 1.149 g | 1.885 g (SHT. NO. 12) |
| 1.0 sec | 0.282 g | 0.572 g (SHT. NO. 11) |

** SINCE ACCELERATION VALUES AS PER LATER DATE AVAILABLE DATA IS LESS THAN 2002 USGS DATA, SEISMIC ANALYSIS IS NOT REVISED.

**Design Seismic Event - 5% Probability of Occurrence in 50 years (975 years Return time)**

**Site Classification** Based on **Seismic** Map

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

TABLE 3.4.2-1 AS PER 2002 USGS DATA

**Bridge location**

Latitude = 36 deg.

Longitude = -89.817 deg. (use -ive value)

As per bridge location & USGS map

**Ss =** 1.885 0.2-second period spectral acceleration (SHT. NO. 12)

**$S_1$ =** 0.572 1-second period spectral acceleration (SHT. NO. 11)

**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

$S_{DS} = Fa*Ss =$ 1.89

$S_{D1} = Fv*S_1 =$ 0.86

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

**RESPONSE SPECTRUM DEVELOPMENT**

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 | 0.75 | $= 0.4S_{DS}$ |
| $T_0 = 0.2*Ts =$ | 0.09 | 1.8852 | $= S_{DS}$ |
| $Ts = S_{D1}/S_{DS} =$ | 0.46 | 1.8852 | $= S_{DS}$ |
| | 0.66 | 1.30 | |
| | 0.86 | 0.99 | |
| | 1.07 | 0.80 | |
| | 1.27 | 0.67 | |
| | 1.48 | 0.58 | |
| | 1.68 | 0.51 | |
| | 1.89 | 0.46 | |
| | 2.09 | 0.41 | |
| | 2.30 | 0.37 | |
| | 2.50 | 0.34 | |

**For a known "T":**

**T =** **1.80**

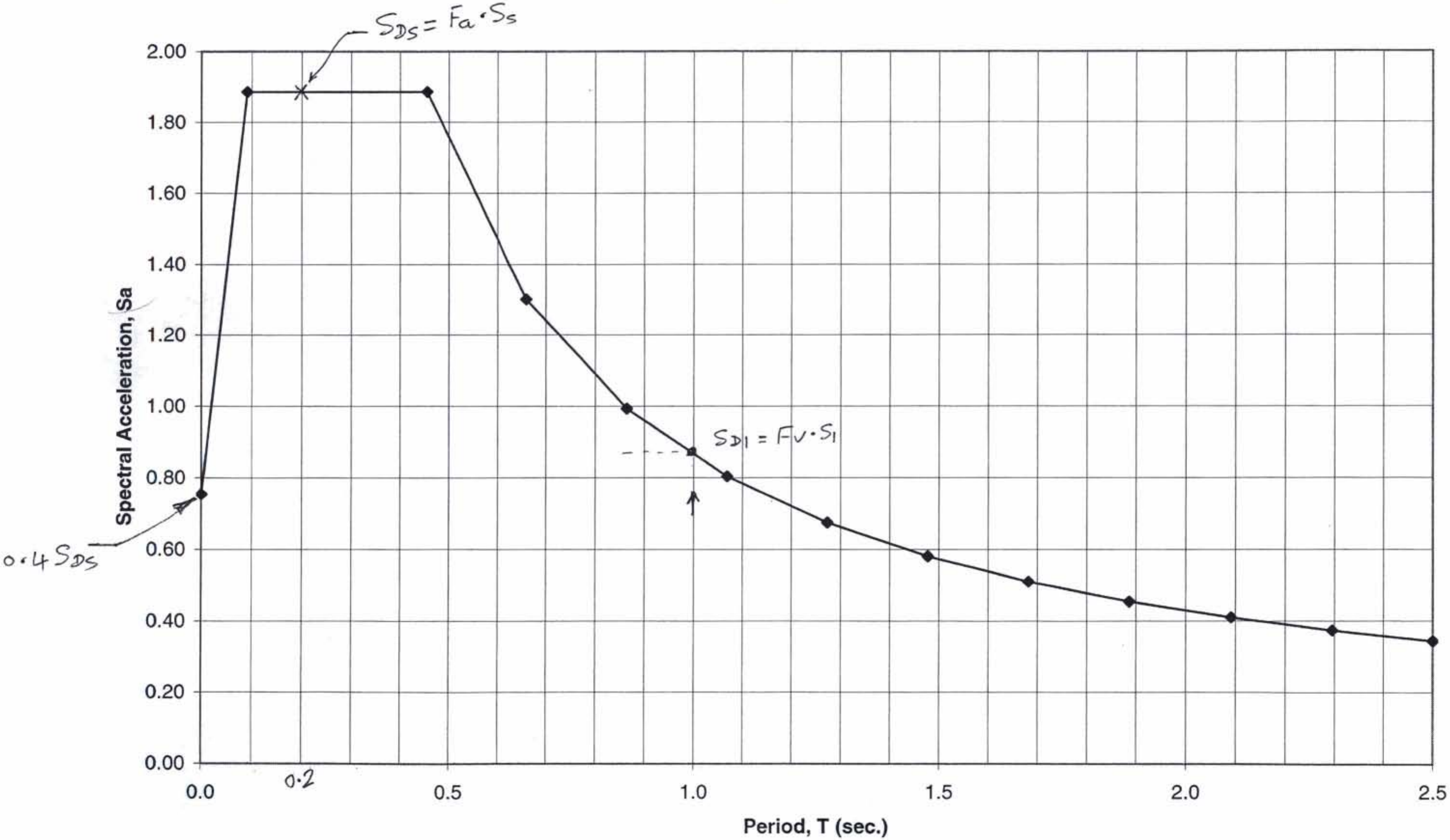
**Sa =** **0.48**

14

SP

EXAMPLE 1 (MO)

**Response Spectrum**



# XTRACT Section Report

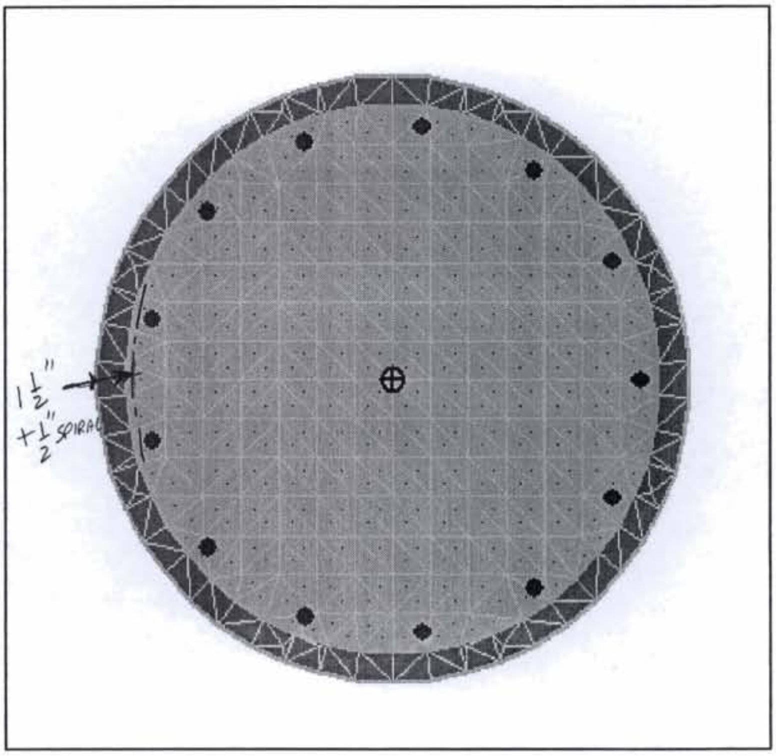
MO DOT 15
MoDOT
2/3/2006
Pemiscot County
Trial 1x
Page __ of __

Section Name: Section1 Mcenter column

EXAMPLE1 (MO) SP

## Section Details:

| | |
|---|---|
| X Centroid: | .1593E-14 in |
| Y Centroid: | -.1457E-16 in |
| Section Area: | 1015 in^2 |
| I gross about X: | 82.90E+3 in^4 |
| I gross about Y: | 82.90E+3 in^4 |
| Reinforcing Bar Area: | 10.21 in^2 |
| Percent Longitudinal Steel: | 1.006 % |
| Overall Width: | 35.93 in |
| Overall Height: | 36.00 in |
| Number of Fibers: | 454 |
| Number of Bars: | 13 |
| Number of Materials: | 3 |




## Material Types and Names:

| | |
|---|---|
| Unconfined Concrete: | Unconfined1 |
| Confined Concrete: | Confined1 |
| Strain Hardening Steel: | Steel1 |

EXAMPLE 1 (MO) SP

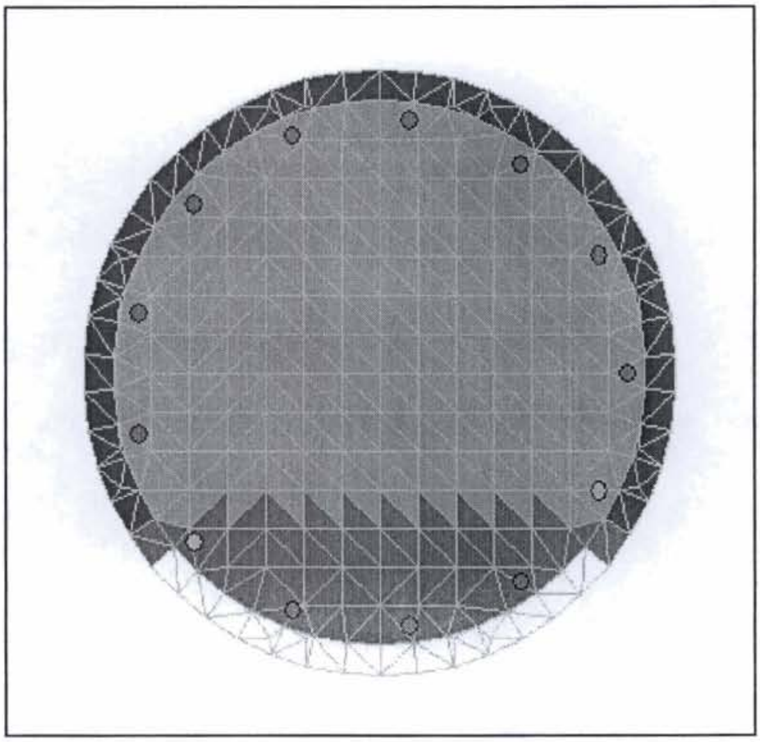
# XTRACT Analysis Report

MO DOT 16
MoDOT
2/3/2006
Pemiscot County
Trial 1x
Page __ of __

| | |
|---|---|
| Section Name: | Section1 |
| Loading Name: | Mcentercol |
| Analysis Type: | Moment Curvature |

## Section Details:

| | |
|---|---|
| X Centroid: | .1328E-15 ft |
| Y Centroid: | -.1214E-17 ft |
| Section Area: | 7.050 ft^2 |

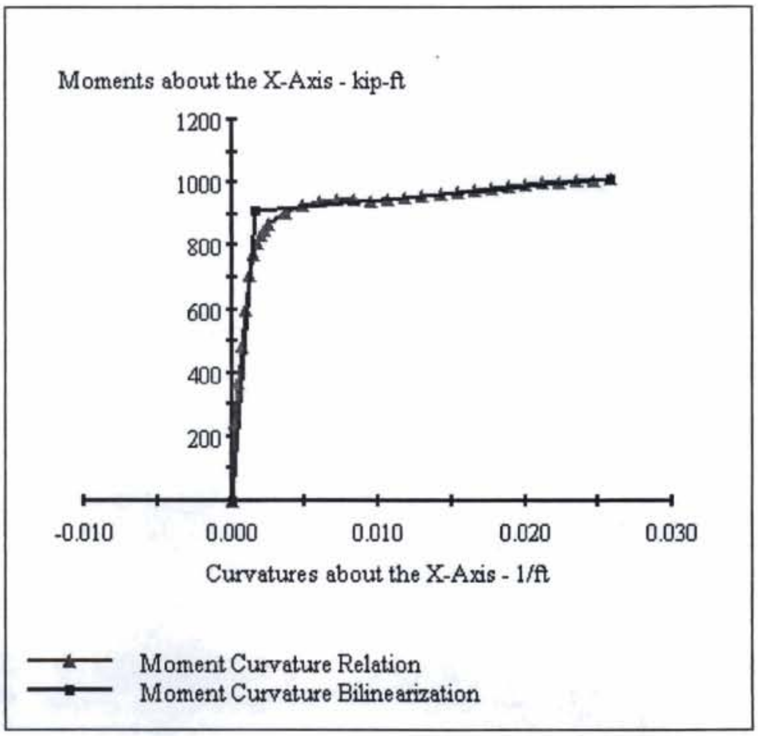


## Loading Details:

| | |
|---|---|
| Constant Load - P: | 177.0 kips ← |
| Incrementing Loads: | Mxx Only |
| Number of Points: | 30 |
| Analysis Strategy: | Displacement Control |

## Analysis Results:

| | |
|---|---|
| Failing Material: | Confined1 |
| Failure Strain: | 15.00E-3 Compression |
| Curvature at Initial Load: | -.4368E-21 1/ft |
| Curvature at First Yield: | 1.266E-3 1/ft |
| Ultimate Curvature: $\phi_u$ | 25.86E-3 1/ft |
| Moment at First Yield: | 706.3 kip-ft |
| Ultimate Moment: | 1014 kip-ft |
| Centroid Strain at Yield: | .6987E-3 Ten |
| Centroid Strain at Ultimate: | 18.13E-3 Ten |
| N.A. at First Yield: | .5521 ft |
| N.A. at Ultimate: | .7012 ft |
| Energy per Length: | 24.04 kips |
| Effective Yield Curvature: | 1.629E-3 1/ft $\phi_y$ |
| Effective Yield Moment: | 909.3 kip-ft ← $M_p$ |
| Over Strength Factor: | 1.116 |
| Plastic Rotation Capacity: | 74.50E-3 rad |
| EI Effective: | 558.1E+3 kip-ft^2 |
| Yield EI Effective: | 4344 kip-ft^2 |
| Bilinear Harding Slope: | .7783 % |
| Curvature Ductility: | 15.87 |




$$N = \frac{N_c \cdot M_p}{(N_c - 1)\,\text{col. spa.}} = \frac{3\,(909.3)}{2 \times 14.33'} = 95.2^k$$

$$P @ \text{ RT col.} = 177 + 95.2 = 272.2^k$$

$$\text{LT col.} = 177 - 95.2 = 81.2^k$$

ExAmplE 1 (MO) SP

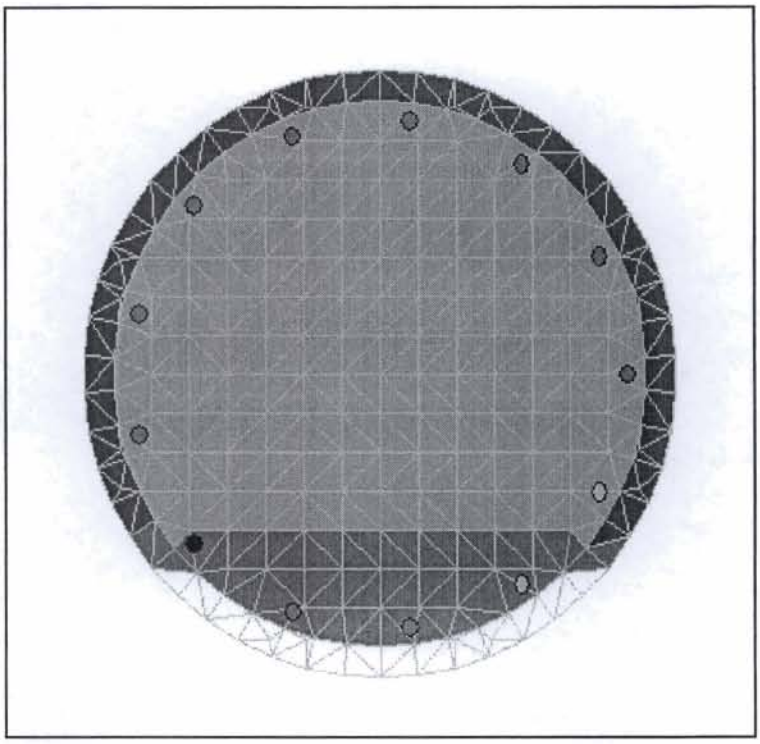
# XTRACT Analysis Report

MO DOT 17
MoDOT
2/9/2006
Pemiscot County
Trial 1x
Page __ of __

| | |
|---|---|
| Section Name: | Section1 |
| Loading Name: | MLeft column |
| Analysis Type: | Moment Curvature |

## Section Details:

| | |
|---|---|
| X Centroid: | 1.646E-6 ft |
| Y Centroid: | -.5828E-16 ft |
| Section Area: | 7.050 ft^2 |

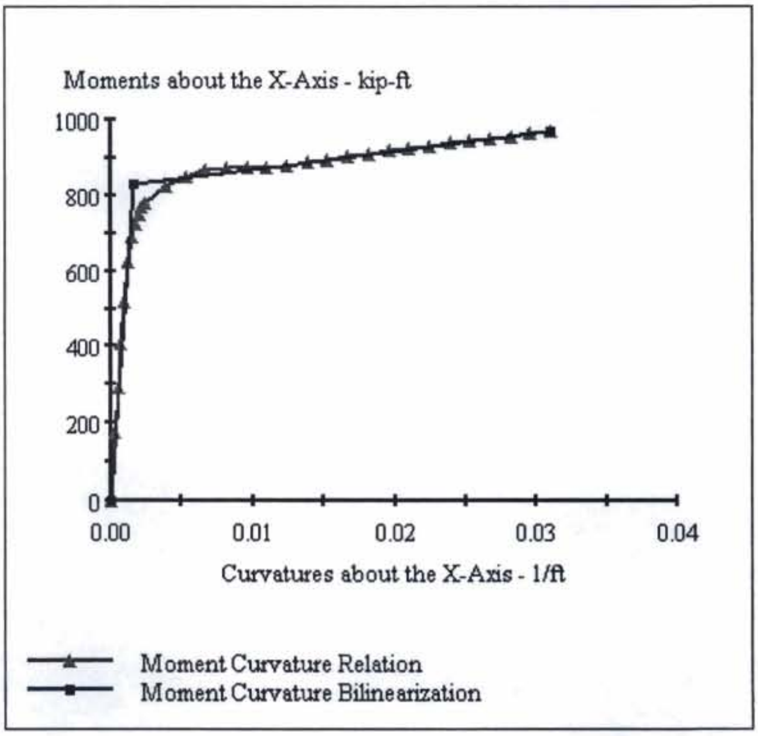


## Loading Details:

| | |
|---|---|
| Constant Load - P: | 81.00 kips |
| Incrementing Loads: | Mxx Only |
| Number of Points: | 30 |
| Analysis Strategy: | Displacement Control |

## Analysis Results:

| | |
|---|---|
| Failing Material: | Confined1 |
| Failure Strain: | 15.00E-3 Compression |
| Curvature at Initial Load: | .8713E-16 1/ft |
| Curvature at First Yield: | 1.213E-3 1/ft |
| Ultimate Curvature: | 31.06E-3 1/ft $\phi u$ |
| Moment at First Yield: | 623.5 kip-ft |
| Ultimate Moment: | 968.5 kip-ft |
| Centroid Strain at Yield: | .7650E-3 Ten |
| Centroid Strain at Ultimate: | 24.77E-3 Ten |
| N.A. at First Yield: | .6308 ft |
| N.A. at Ultimate: | .7974 ft |
| Energy per Length: | 27.12 kips |
| Effective Yield Curvature: | 1.611E-3 1/ft $\phi y$ |
| Effective Yield Moment: | 828.1 kip-ft |
| Over Strength Factor: | 1.170 |
| Plastic Rotation Capacity: | 86.73E-3 rad |
| EI Effective: | 514.1E+3 kip-ft^2 |
| Yield EI Effective: | 4768 kip-ft^2 |
| Bilinear Harding Slope: | .9275 % |
| Curvature Ductility: | 19.28 |



EXAMPLE 1 (MO) SP

# XTRACT Analysis Report

MO DOT 18
MoDOT
2/9/2006
Pemiscot County
Trial 1x
Page __ of __

| | |
|---|---|
| Section Name: | Section1 |
| Loading Name: | MRt column |
| Analysis Type: | Moment Curvature |

## Section Details:

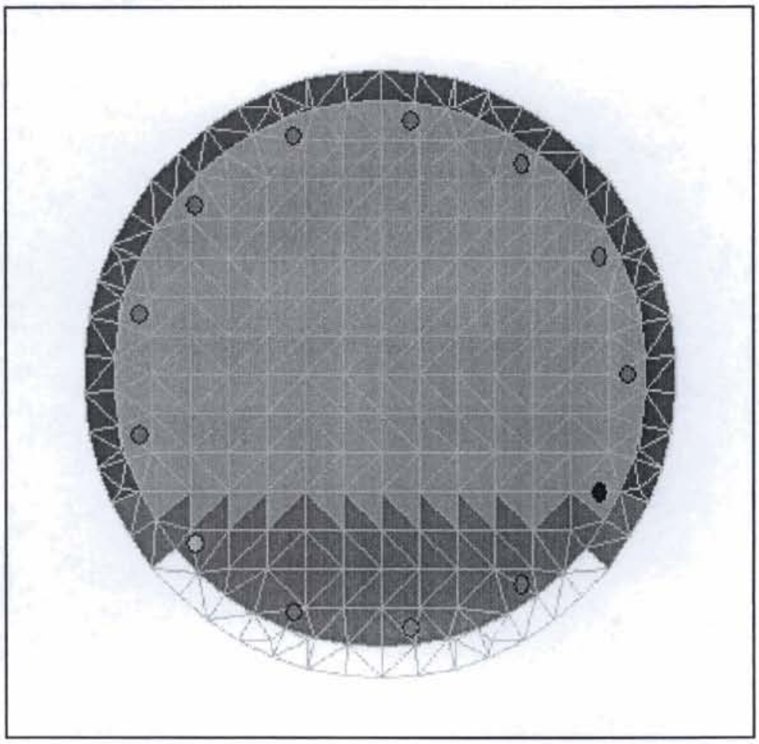
| | |
|---|---|
| X Centroid: | 1.646E-6 ft |
| Y Centroid: | -.5828E-16 ft |
| Section Area: | 7.050 ft^2 |

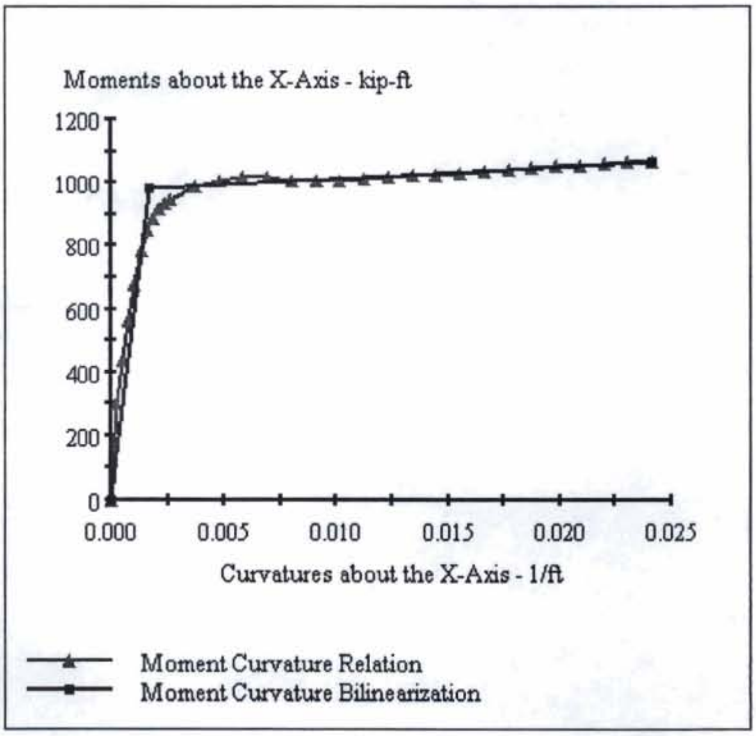
## Loading Details:

| | |
|---|---|
| Constant Load - P: | 272.0 kips ✓ |
| Incrementing Loads: | Mxx Only |
| Number of Points: | 30 |
| Analysis Strategy: | Displacement Control |

## Analysis Results:

| | |
|---|---|
| Failing Material: | Confined1 |
| Failure Strain: | 15.00E-3 Compression |
| Curvature at Initial Load: | .1393E-14 1/ft |
| Curvature at First Yield: | 1.318E-3 1/ft |
| Ultimate Curvature: | 24.18E-3 1/ft |
| Moment at First Yield: | 783.8 kip-ft |
| Ultimate Moment: | 1070 kip-ft |
| Centroid Strain at Yield: | .6332E-3 Ten |
| Centroid Strain at Ultimate: | 15.96E-3 Ten |
| N.A. at First Yield: | .4803 ft |
| N.A. at Ultimate: | .6600 ft |
| Energy per Length: | 23.91 kips |
| Effective Yield Curvature: | 1.652E-3 1/ft |
| Effective Yield Moment: | 982.0 kip-ft ✓ |
| Over Strength Factor: | 1.089 |
| Plastic Rotation Capacity: | 66.33E-3 rad |
| EI Effective: | 594.5E+3 kip-ft^2 |
| Yield EI Effective: | 3886 kip-ft^2 |
| Bilinear Harding Slope: | .6536 % |
| Curvature Ductility: | 14.63 |





(P/Pu)^2 + 3431*(P/Pu)^3

EXAMPLE 1 (MO)　　SP　　19

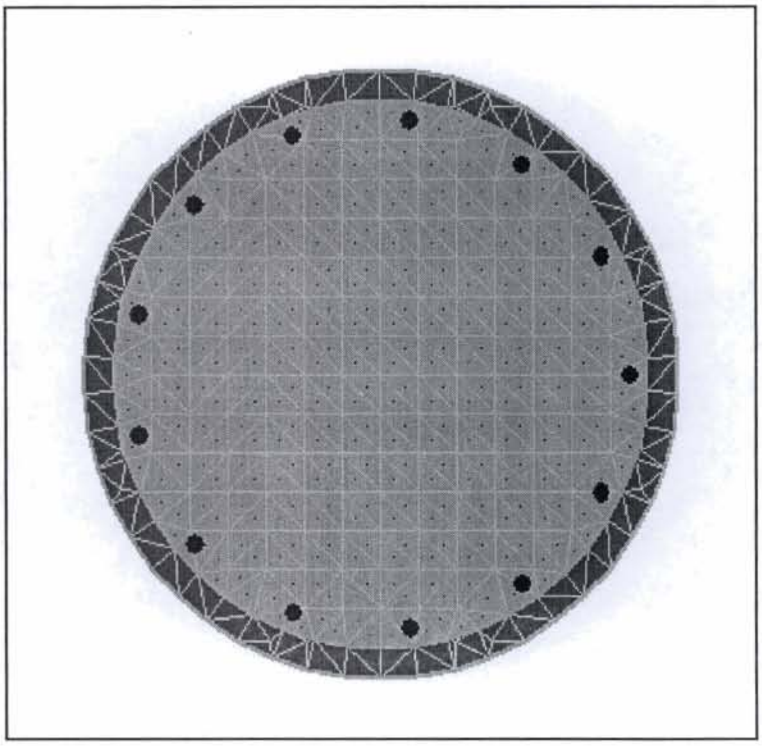
# XTRACT Analysis Report

MO DOT
MoDOT
2/3/2006
Pemiscot County
Trial 1x
Page __ of __

| | |
|---|---|
| Section Name: | Section1 |
| Loading Name: | Mcenter column |
| Analysis Type: | PM Interaction |

## Section Details:

| | |
|---|---|
| X Centroid: | .1328E-15 ft |
| Y Centroid: | -.1214E-17 ft |
| Section Area: | 7.050 ft^2 |

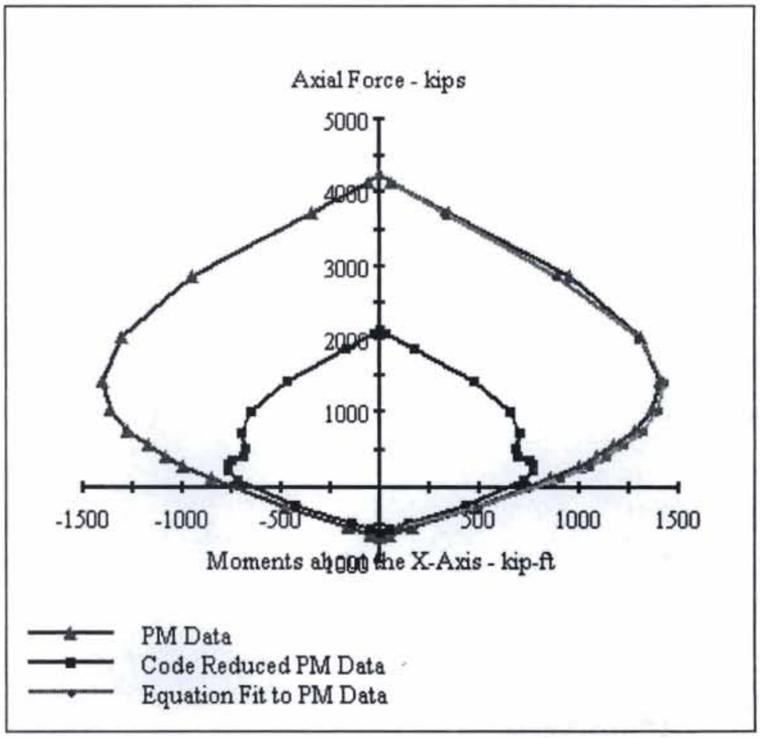


## Loading Details:

| | |
|---|---|
| Angle of Loading: | 0 deg |
| Number of Points: | 40 |
| Min. Unconfined1 Strain: | 3.000E-3 Comp |
| Max. Unconfined1 Strain: | 1.0000 Ten |
| Min. Confined1 Strain: | 5.000E-3 Comp |
| Max. Confined1 Strain: | 1.0000 Ten |
| Min. Steel1 Strain: | 8.000E-3 Comp |
| Max. Steel1 Strain: | 8.000E-3 Ten |

## Analysis Results:

| | |
|---|---|
| Max. Compression Load: | 4264 kips |
| Max. Tension Load: | -673.9 kips |
| Maximum Moment: | 1402 kip-ft |
| P at Max. Moment: | 1429 kips |
| Minimum Moment: | -1402 kip-ft |
| P at Min. Moment: | 1429 kips |
| Moment (Mxx) at P=0: | 739.7 kip-ft |
| Max. Code Comp. Load: | 2132 kips |
| Max. Code Ten. Load: | -606.5 kips |
| Maximum Code Moment: | 768.5 kip-ft |
| P at Max. Code Moment: | 293.2 kips |
| Minimum Code Moment: | -768.5 kip-ft |
| P at Min. Code Moment: | 293.2 kips |
| PM Interaction Equation: | Units in kip-ft |




CODE REDUCTION AS PER
LRFD P. 5-126 @ 0.2 $A_g f'_c$ →

$\phi c = 0.5$

$\phi b = 0.5$

EXAMPLE1 (MO) SP

# XTRACT Analysis Report

MO DOT 20
MoDOT
2/3/2006
Pemiscot County
Trial 1x
Page __ of __

Section Name: Section1
Loading Name: Mcenter column
Analysis Type: PM Interaction

| Code P (kips) | Code M (kip-ft) | |
|---|---|---|
| 2132 | ~~-.3996E-13~~ | 0.1E-10 |
| 2069 | 29.59 | |
| 1859 | 171.6 | |
| 1434 | 475.5 | |
| 1012 | 655.1 | |
| 714.5 | 701.1 | |
| 513.3 | 682.9 | |
| 412.4 | 695.1 | |
| 355.8 | 751.9 | |
| 293.2 | 768.5 | |
| 220.5 | 767.2 | |
| 101.2 | 725.5 | |
| -231.4 | 433.7 | |
| -478.0 | 147.6 | |
| -568.1 | 45.43 | |
| -606.5 | ~~.2195E-12~~ | 0.1E-10 |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -606.5 | .2195E-12 | |
| -568.1 | -45.43 | |
| -478.0 | -147.6 | |
| -231.3 | -433.8 | |
| 101.3 | -725.6 | |
| 220.6 | -767.2 | |

EXAMPLE1 (MO)

# CAPP Project Report

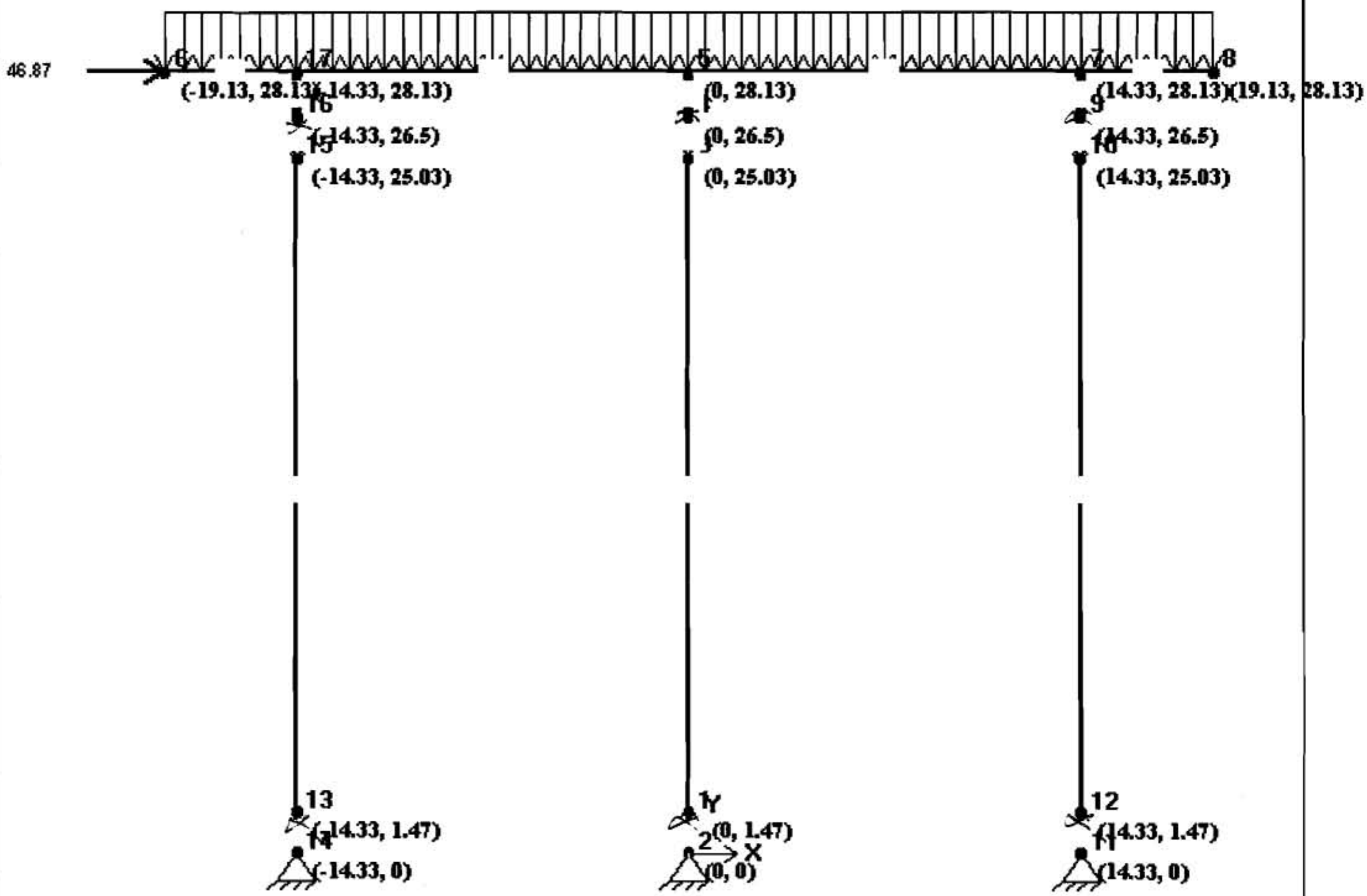
SPatel 21
H Model.cap
2/16/2006

Loading Name: Combo1

Report Type: Undeformed Shape - Units: ft

Comments: LRFD Guide 2005 Seismic Example1

Page __ of __



```
# CAPP Generated Input file
# Created = 2/16/2006

UNITS
     FORCE=Kips
     LENGTH=Feet

JOINT
     NUM=1  X=0  Y=1.47
     NUM=2  X=0  Y=0
     NUM=3  X=0  Y=25.03
     NUM=4  X=0  Y=26.5
     NUM=5  X=0  Y=28.13
     NUM=6  X=-19.13  Y=28.13
     NUM=7  X=14.33  Y=28.13
     NUM=8  X=19.13  Y=28.13
     NUM=9  X=14.33  Y=26.5
     NUM=10  X=14.33  Y=25.03
     NUM=11  X=14.33  Y=0
     NUM=12  X=14.33  Y=1.47
     NUM=13  X=-14.33  Y=1.47
     NUM=14  X=-14.33  Y=0
     NUM=15  X=-14.33  Y=25.03
     NUM=16  X=-14.33  Y=26.5
     NUM=17  X=-14.33  Y=28.13

RESTRAINT
     NUM=2 DOF=U1,U2
     NUM=11 DOF=U1,U2
     NUM=14 DOF=U1,U2

MATERIAL
     NAME=Mat1  TYPE=Concrete  E=545.2E+3  W=0.15

SECTION
     NAME=Section1  TYPE=User_Defined  MAT=Mat1  I=0.94  A=7.07
     NAME=Section2  TYPE=User_Defined  MAT=Mat1  I=1.02  A=7.07
     NAME=Section3  TYPE=User_Defined  MAT=Mat1  I=1.09  A=7.07
     NAME=Section4  TYPE=User_Defined  MAT=Mat1  I=5.01  A=11.38

ELEMENT
     NAME=Element1  TYPE=Elastic_Beam_Column  SEC=Section1
     NAME=Element2  TYPE=Elastic_Beam_Column  SEC=Section2
     NAME=Element5  TYPE=Rigid_Link
     NAME=Element3  TYPE=Elastic_Beam_Column  SEC=Section3
     NAME=Element4  TYPE=Elastic_Beam_Column  SEC=Section4

HINGE
     NAME=Hinge1  TYPE=Interaction_Hinge  ROTCAP=86.73E-3  CLIMIT=2132  TLIMIT=-606.5  PU=2132
HINGE_MODE=2
          DATA  P=-606.5  M=1.E-9
          DATA  P=-568.1  M=45.43
          DATA  P=-478  M=147.6
          DATA  P=-231.4  M=433.7
          DATA  P=101.2  M=725.5
          DATA  P=220.5  M=767.2
          DATA  P=293.2  M=768.5
          DATA  P=355.8  M=751.9
          DATA  P=412.4  M=695.1
          DATA  P=513.3  M=682.9
          DATA  P=714.5  M=701.1
          DATA  P=1012  M=655.1
          DATA  P=1434  M=475.5
          DATA  P=1859  M=171.6
          DATA  P=2069  M=29.59
          DATA  P=2132  M=1.E-9
     NAME=Hinge2  TYPE=Interaction_Hinge  ROTCAP=74.5E-3  CLIMIT=2132  TLIMIT=-606.5  PU=2132
HINGE_MODE=2
          DATA  P=-606.5  M=1.E-9
          DATA  P=-568.1  M=45.43
          DATA  P=-478  M=147.6
          DATA  P=-231.4  M=433.7
          DATA  P=101.2  M=725.5
          DATA  P=220.5  M=767.2
          DATA  P=293.2  M=768.5
```

```
        DATA  P=355.8  M=751.9
        DATA  P=412.4  M=695.1
        DATA  P=513.3  M=682.9
        DATA  P=714.5  M=701.1
        DATA  P=1012  M=655.1
        DATA  P=1434  M=475.5
        DATA  P=1859  M=171.6
        DATA  P=2069  M=29.59
        DATA  P=2132  M=1.E-9
    NAME=Hinge3  TYPE=Interaction_Hinge  ROTCAP=66.33E-3  CLIMIT=2132  TLIMIT=-606.5  PU=2132
HINGE_MODE=2
        DATA  P=-606.5  M=1.E-9
        DATA  P=-568.1  M=45.43
        DATA  P=-478  M=147.6
        DATA  P=-231.4  M=433.7
        DATA  P=101.2  M=725.5
        DATA  P=220.5  M=767.2
        DATA  P=293.2  M=768.5
        DATA  P=355.8  M=751.9
        DATA  P=412.4  M=695.1
        DATA  P=513.3  M=682.9
        DATA  P=714.5  M=701.1
        DATA  P=1012  M=655.1
        DATA  P=1434  M=475.5
        DATA  P=1859  M=171.6
        DATA  P=2069  M=29.59
        DATA  P=2132  M=1.E-9

MEMBER
    NUM=1  ELEM=Element1  INODE=14  JNODE=13  JHINGE=Hinge1
    NUM=2  ELEM=Element1  INODE=13  JNODE=15
    NUM=3  ELEM=Element1  INODE=15  JNODE=16  IHINGE=Hinge1
    NUM=4  ELEM=Element5  INODE=16  JNODE=17
    NUM=5  ELEM=Element5  INODE=9  JNODE=7
    NUM=6  ELEM=Element3  INODE=11  JNODE=12  JHINGE=Hinge3
    NUM=7  ELEM=Element3  INODE=12  JNODE=10
    NUM=8  ELEM=Element3  INODE=10  JNODE=9  IHINGE=Hinge3
    NUM=9  ELEM=Element2  INODE=2  JNODE=1  JHINGE=Hinge2
    NUM=10  ELEM=Element2  INODE=1  JNODE=3
    NUM=11  ELEM=Element4  INODE=7  JNODE=8
    NUM=12  ELEM=Element2  INODE=3  JNODE=4  IHINGE=Hinge2
    NUM=15  ELEM=Element5  INODE=4  JNODE=5
    NUM=17  ELEM=Element4  INODE=5  JNODE=7
    NUM=18  ELEM=Element4  INODE=17  JNODE=5
    NUM=19  ELEM=Element4  INODE=6  JNODE=17

LOAD
    NAME=Load1  TYPE=Dead_Load
        MEMDATA  NUM=11  IY=-13.86  JY=-13.86 K/ft
        MEMDATA  NUM=17  IY=-13.86  JY=-13.86
        MEMDATA  NUM=18  IY=-13.86  JY=-13.86
        MEMDATA  NUM=19  IY=-13.86  JY=-13.86
    NAME=Load2  TYPE=Push_Load
        NODEDATA  NUM=6  X=10

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

GRIDLINES

PROJECT_PROPERTIES
    DESCRIPTION=LRFDseismic Example1

END
```

Example 1 (MO) SP 24

# CAPP Analysis Report

SPatel
H Model.cap
2/16/2006

Loading Name: Combo1

Report Type: Push Over Analysis Summary

Comments: LRFD Guide 2005 Seismic Examlpe1

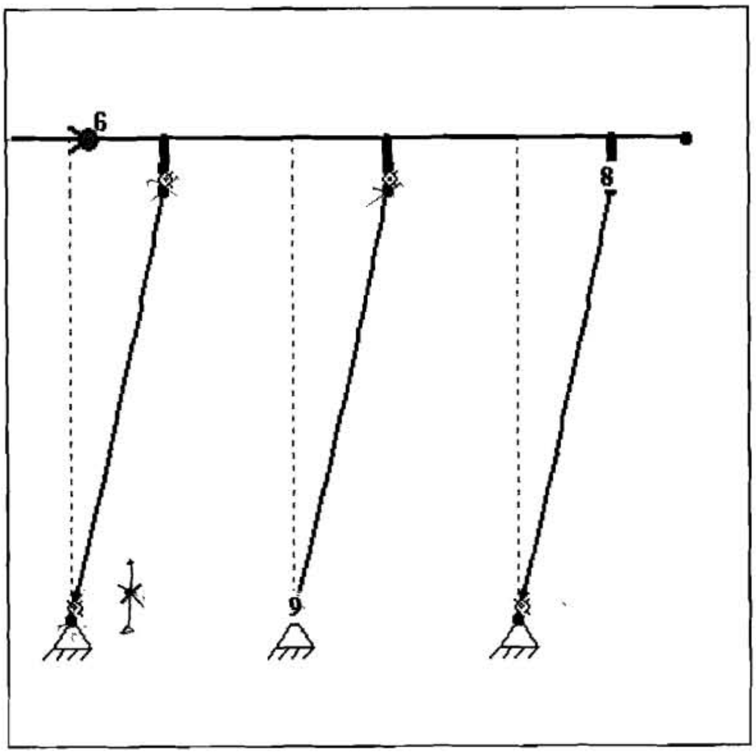
Page __ of __

## Model Details:

Nmber of Members/Nodes: 16 Members, 17 Nodes

Overall Width: 38.26 ft

Overall Height: 28.13 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. 8

Member Element Type: Element3 - Beam Column

Termination Cause: Interact. Hinge - Hinge3

Hinge3 : 66.33E-3 rad

Last Hinge Moment: 767.6 kips-ft

Mem. Drift at Termination: 0.1911%

## Analysis Results:

Critical Node (Node Shown): 6

Number of Events: 7

First X Yield Push Load: 79.03 kips

Max X Push Load: 81.37 kips

Last X Push Load: 46.87 kips

X First Yield Displacement: 0.3421 ft

X Ultimate Displacement: 2.007 ft

Area Under Push-Disp Curve: 120.8 kips-ft

Effective Yield Disp: 0.3539 ft

Effective Yield Push Load: 81.75 kips

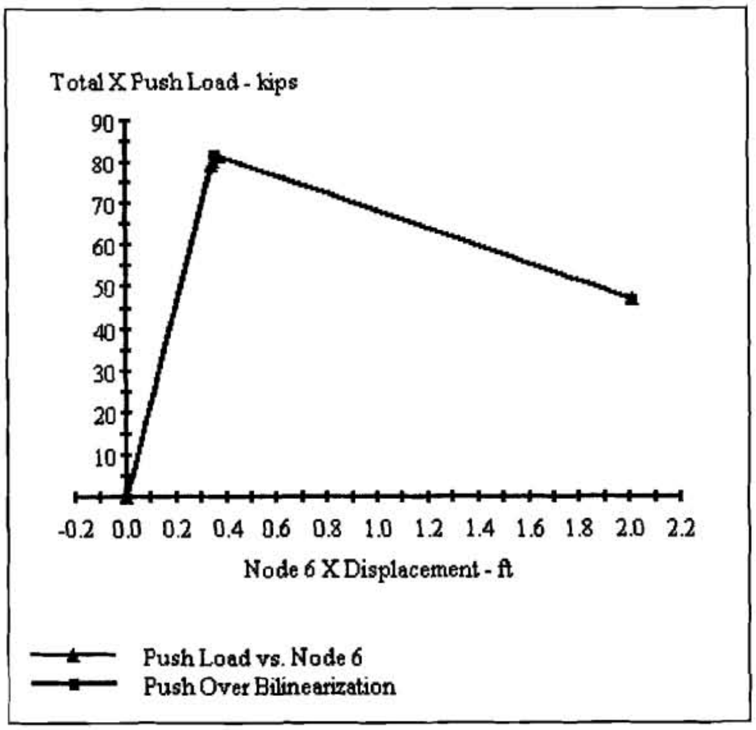
Eff System Ductility: 5.67

Eff Elastic Stiffness: 230.9 kips/ft

Eff Plastic Stiffness: 21.1 kips/ft

Bilinear Harding Slope: -9.141 %

Over Strength Factor: 0.5734



EXAMPLE1 (MO)

SP

25

# CAPP Analysis Report

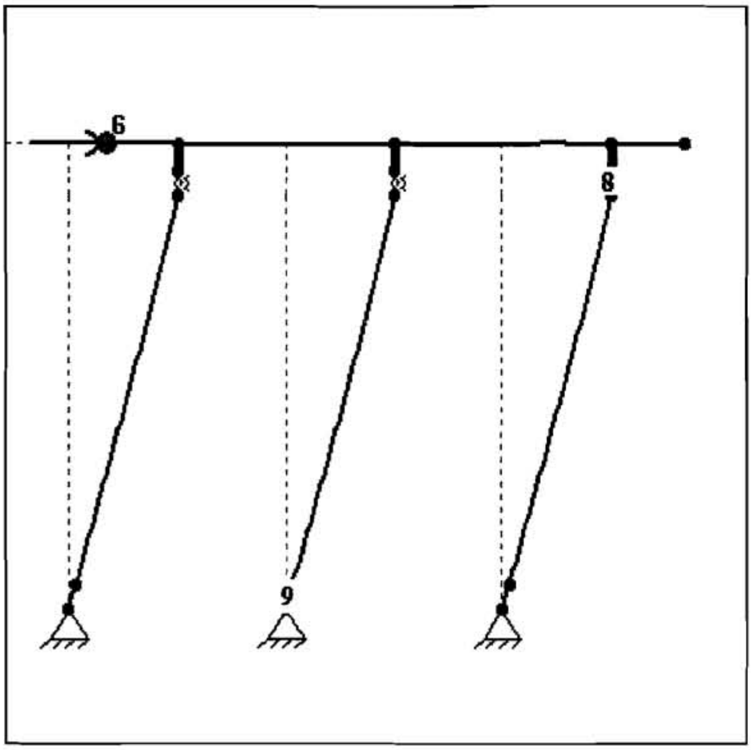
SPatel
Pinmodel.cap
2/16/2006

| | |
|---|---|
| Loading Name: | Combo1 |
| Report Type: | Push Over Analysis Summary |
| Comments: | LRFD Guide 2005 Seismic Example1 (w/o Hinge @ bottom) |

Page __ of __

## Model Details:

| | |
|---|---|
| Nmber of Members/Nodes: | 16 Members, 17 Nodes |
| Overall Width: | 38.26 ft |
| Overall Height: | 28.13 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 |

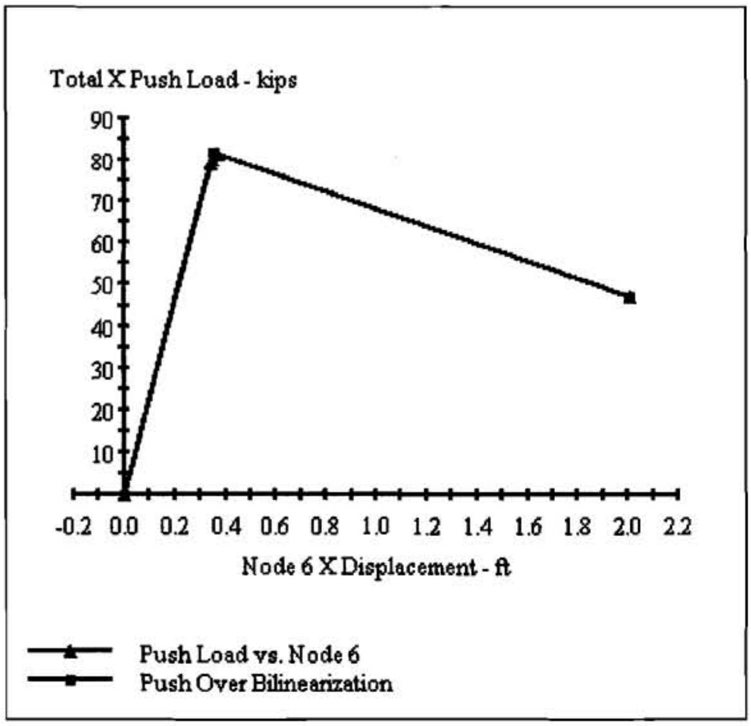
## Termination Details:

| | |
|---|---|
| Analysis Termination: | Cap. Reached in Mem. 8 |
| Member Element Type: | Element3 - Beam Column |
| Termination Cause: | Interact. Hinge - Hinge3 |
| Hinge3 : | 66.33E-3 rad |
| Last Hinge Moment: | 767.6 kips-ft |
| Mem. Drift at Termination: | 0.1911% |

NOTE: RESULTS ARE SAME AS PREVIOUS SHEET

## Analysis Results:

| | |
|---|---|
| Critical Node (Node Shown): | 6 |
| Number of Events: | 7 |
| First X Yield Push Load: | 79.03 kips |
| Max X Push Load: | 81.37 kips |
| Last X Push Load: | 46.87 kips |
| X First Yield Displacement: | 0.3421 ft |
| X Ultimate Displacement: | 2.007 ft |
| Area Under Push-Disp Curve: | 120.8 kips-ft |
| Effective Yield Disp: | 0.3539 ft |
| Effective Yield Push Load: | 81.75 kips |
| Eff System Ductility: | 5.67 |
| Eff Elastic Stiffness: | 230.9 kips/ft |
| Eff Plastic Stiffness: | 21.1 kips/ft |
| Bilinear Harding Slope: | -9.141 % |
| Over Strength Factor: | 0.5734 |



X-TRACT INPUT & SHEAR DEMAND TO CAPACITY RATIO

Total superstruct Dead Load $DL_{super} := 530$ kip

Column Information

column dia. $D := 3.0$ ft $L := 26.5$ ft

Conc. density $\rho := 150$ PCF $Ag := \left(\frac{\pi}{4}\right)(D \cdot 12)^2$ $Ag = 1018$ sq. in

No. of column $Nc := 3$ $Column_{spacing} := 14.33$ ft

Total Column DL $WcDL := Nc \cdot \frac{\rho}{1000} \cdot L \cdot \frac{Ag}{144}$ $WcDL = 84$ kips

Beam Length $B_L := 38.2$ ft $B_w := 3.50$ ft $B_h := 3.2$ ft

$BM_{area} := B_w \cdot B_h$ **Beam information assumed** $\boxed{BM_{area} = 11.4}$ $ft^2$

Beam DL $BM_{DL} := BM_{area} \cdot B_L \cdot \frac{\rho}{1000}$ $BM_{DL} = 65$ kips

**DL per column** $P := \frac{DL_{super}}{Nc}$ $\boxed{P = 177}$ Kips /column

Uniform DL @ beam without BM DL $w := \frac{DL_{super}}{B_L}$ $\boxed{w = 13.86}$ k /ft

**Use Expected concrete strength = Fc' * 1.3 <=5 ksi** Eqn 8.7)

$Fcbeam := 3.9$ ksi for beam

$$Ecbeam := \frac{\left(\rho^{1.5} \cdot 33 \cdot \sqrt{Fcbeam \cdot 1000}\right) \cdot 144}{1000}$$ $\boxed{Ecbeam = 545187}$ ksf

$Fccol := 3.9$ ksi for column

$$Eccol := \frac{\left(\rho^{1.5} \cdot 33 \cdot \sqrt{Fccol \cdot 1000}\right) \cdot 144}{1000}$$ $\boxed{Eccol = 545187}$ ksf

***Run Xtract*** to find effective yield Moment (plastic moment), ***Mp*** of the bent using DL per column

**Calculate Elastic Stiffness** ***Center column***

Effective Yield Mom, $M_p$ $M_p := 909.3$ k – ft

Effective Yield Curvature, $\phi_y$ $\phi_y := 0.001629$ 1 /ft

$EI_{eff} := \frac{M_p}{\phi_y}$ Eqn 5.4 **Center col** $EI_{eff} = 558195$ k /ft^2

$K := \frac{3 \cdot EI_{eff}}{L^3}$ $K = 90$ k /ft

**Calculate axial force due to overturning**

$$N := \frac{Nc \cdot M_p}{(Nc - 1)Column_{spacing}} \qquad N = 95 \quad \text{kips}$$

**Left column** $Pleft := -N + P$ **Rt column** $Prt := N + P$

***Rerun Xtract with new axial loads*** to find column Icrack & Hinge properties

**Left column** $\boxed{Pleft = 81}$ **kips** **Center col** $\boxed{P = 177}$ **kips** **Rt column** $\boxed{Prt = 272}$ **kips**

$LeftEI_{eff} := 514100$ k /ft^2 $\quad LeftIcol_{crack} := \frac{LeftEI_{eff}}{Eccol} \quad \boxed{LeftIcol_{crack} = 0.94}$ $ft^4$

$EI_{eff} = 558195$ k /ft^2 $\quad CentIcol_{crack} := \frac{EI_{eff}}{Eccol} \quad \boxed{CentIcol_{crack} = 1.02}$ $ft^4$

$RtEI_{eff} := 594500$ k /ft^2 $\quad RtIcol_{crack} := \frac{RtEI_{eff}}{Eccol} \quad \boxed{RtIcol_{crack} = 1.09}$ $ft^4$

***Calculate Plastic Hinge Length** or **use Xtract program option to compute Hinge length***

$f_y := 60$ **ksi** column long. bar diam. $d_b := 1.0$ **inch** $\quad f_{ye} := f_y \cdot 1.1$

$$l_{p1} := \frac{0.08 \cdot L \cdot 12 + 6 \cdot \frac{f_{ye}}{40} \cdot d_b}{12} \qquad l_{pmin} := 0.3 \cdot f_{ye} \cdot \frac{d_b}{12}$$

ART. 4.11.6

$l_{p1} = 2.9$ ft $\quad l_{pmin} = 1.7$ ft

Plastic Hinge Length, $l_p$ $\quad l_p := \max(l_{p1}, l_{pmin}) \quad \boxed{l_p = 2.95}$ ft $\quad \boxed{\frac{l_p}{2} = 1.47}$ ft

***Calculation of Ductility Capacity***

Calculate plastic shear Vp, Yield Deflections and Stiffness for each column.

| **Left column** | **Center Column** | **Right Column** |
|---|---|---|
| $K_L := 3\frac{LeftEI_{eff}}{L^3}$ | $K_C := K$ | $K_R := 3\frac{RtEI_{eff}}{L^3}$ |
| $K_L = 83$ k /ft | $K_C = 90$ k /ft | $K_R = 96$ k /ft |
| $M_{pLeft} := 828.1$ k – ft | $M_p = 909.3$ k – ft | $M_{pRight} := 982$ k – ft |
| $M_{pL} := M_{pLeft} \cdot 1.2$ | $M_{pC} := M_p \cdot 1.2$ | $M_{pR} := M_{pRight} \cdot 1.2$ |
| $V_{pLeft} := \frac{M_{pL}}{L}$ | $V_{pCenter} := \frac{M_{pC}}{L}$ | $V_{pRight} := \frac{M_{pR}}{L}$ |
| $V_{pLeft} = 37.5$ | $V_{pCenter} = 41.2$ | $V_{pRight} = 44.5$ |
| $\Delta_{yLeft} := \frac{V_{pLeft} \cdot 12}{K_L}$ | $\Delta_{yCenter} := \frac{V_{pCenter} \cdot 12}{K_C}$ | $\Delta_{yRight} := \frac{V_{pRight} \cdot 12}{K_R}$ |

$\Delta_{yLeft} = 5.4$ inch $\Delta_{yCenter} = 5.5$ inch $\Delta_{yRight} = 5.6$ inch

Calculate Plastic Displacement for Each Column SECT 4.8
Ultimate Curvature, $\phi_u$ Effective Yield Curvature, $\phi_y$

**Left Column**

$\phi_{uL} := 0.03106$ 1 /ft

$\phi yL := 0.001611$ 1 /ft

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

$\theta_{pL} = 0.0867$

$\Delta_{pL} := 12\theta_{pL} \cdot \left(L - \frac{l_p}{2}\right)$

$\Delta_{pL} = 26$ in

**Center Column**

$\phi_{uC} := 0.02586$ 1 /ft

$\phi yC := 0.001629$ 1 /ft

$\theta_{pC} := (\phi_{uC} - \phi yC) \cdot l_p$

$\theta_{pC} = 0.0714$

$\Delta_{pC} := 12\theta_{pC} \cdot \left(L - \frac{l_p}{2}\right)$

$\Delta_{pC} = 21.4$ in

**Right Column**

$\phi_{uR} := 0.02418$ 1 /ft

$\phi yR := 0.001652$ 1 /ft

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

$\theta_{pR} = 0.0663$

$\Delta_{pR} := 12\theta_{pR} \cdot \left(L - \frac{l_p}{2}\right)$

$\Delta_{pR} = 19.9$ in

Displacement capacity, $\Delta$

$\Delta_{uL} := \Delta_{pL} + \Delta_{yLeft}$ $\Delta_{uC} := \Delta_{pC} + \Delta_{yCenter}$ $\Delta_{uR} := \Delta_{pR} + \Delta_{yRight}$

$\Delta_{uL} = 31.5$ in $\Delta_{uC} = 26.9$ in $\Delta_{uR} = 25.5$ in

$\Delta_u := \min(\Delta_{uL}, \Delta_{uR}, \Delta_{uC})$

<u>**Displacement Capacity**</u> $\Delta_u = 25.49$ in

**Period of the Bent in Transverse Direction:**

**Use Substruct DL multiplier = 1** $Fcator_{DL} := 1$

Total Column DL $WcDL = 84$ kips Beam DL $BM_{DL} = 65.3$ kips

Dead load = DL superstruct + substrDL $substrDL := (WcDL + BM_{DL}) \cdot Fcator_{DL}$

Bent Dead Load $WDL := DL_{super} + substrDL$ $WDL = 680$ kips

Total Stiffness: $K_T := K_L + K_R + K_C$ $K_T = 269$ k /ft

$\omega := \left(32.2 \cdot \frac{K_T}{WDL}\right)^{0.5}$ $\omega = 3.6$ $T1 := 2 \cdot \frac{\pi}{\omega}$ $T1 = 1.8$ sec

**Use value for Sa @ T period** from Response spectra development

$Sa := 0.48$

**Displacement Demand - Capacity Ratio**

$\Delta_d := 12WDL \cdot \left(\frac{Sa}{K_T}\right)$ $\Delta_d = 14.57$ **inch**

Establish Demand to Capacity Ratio, R = Δdemand / Δcapacity

$R1 := \frac{\Delta_d}{\Delta_u}$ $R1 = 0.6$

$$Rratio1 := \begin{cases} "O.K." & \text{if } R1 \le 1 \\ "N.G." & \text{otherwise} \end{cases}$$

Rratio1 = "O.K."

Displacement ductility demand for each column, $\mu_D$ = Δdemand / Δy

| Left column | Center column | Right column |
|---|---|---|
| $\mu_L := \frac{\Delta_d}{\Delta_{yLeft}}$ | $\mu_C := \frac{\Delta_d}{\Delta_{yCenter}}$ | $\mu_R := \frac{\Delta_d}{\Delta_{yRight}}$ |
| $\mu_L = 2.7$ | $\mu_C = 2.7$ | $\mu_R = 2.6$ |

$Max\ \mu_D := \max(\mu_L, \mu_C, \mu_R)$ $Max\ \mu_D = 2.7$

**Target Displacement Ductility Demand, μ**

| **New Design** ART 4.9 | | **Retrofit** | |
|---|---|---|---|
| Single Column Bent | $\mu_D$<=6 | Single Column Bent | $\mu_D$<=6 |
| Multi Column Bent | $\mu_D$<=8 | Multi Column Bent | $\mu_D$<=8 |

Use maximum $\mu_{Dallowed} := 8$

$$\mu_DRatio := \begin{cases} "O.K." & \text{if } \mu_L < \mu_{Dallowed} \wedge \mu_R < \mu_{Dallowed} \wedge \mu_C < \mu_{Dallowed} \\ "N.G." & \text{otherwise} \end{cases}$$

$\mu_D$Ratio = "O.K."

***Shear Demand to capacity ratio*** ART 8.6

1. calculate $\rho f_y$

**Spiral bar** information **Try #4 @ 3"** instead of #5 @ 3" pitch

$f_{yt} := 60$ ksi $A_{bt} := 0.196$ sq. in $s := 3$ in $c := 1.5$ inch conc. cover

$$\rho_s := 4.A_{bt}\cdot\frac{1}{(D\cdot 12 - 2\cdot c)\cdot s}$$

$\rho_{smin} := 0.004$ For SDC D or C **0.4%**
For SDC B **0.2%** as per **8.6.6**

$$\rho_s = 0.0079$$

$$\rho_{s1} := \begin{cases} \text{" O.K. "} & \text{if } \rho_{smin} < \rho_s \\ \text{" N.G. "} & \text{otherwise} \end{cases}$$

Shear reinf $\rho_{s1} = \text{" O.K. "}$

2. Calculate Vc **(Inside the plastic Hinge zone)** ART 8.6.2 Page 8-9

$Ae := 0.8\cdot Ag$ $Ae = 814$ sq in $SDC := 4$ For SDC = D use 4
SDC = C use 3

$$\alpha 1 := \begin{cases} 0.010\rho_s\cdot f_{yt} & \text{if } SDC = 3 \\ 0.03\cdot\rho_s\cdot\frac{f_{yt}\cdot 1000}{\mu_{\mathcal{D}}} & \text{otherwise} \end{cases}$$

Replace info for C & B if SDC = B
$0.015\rho_s\cdot f_{yt}$ if $SDC = 2$ For SDC = B use 2

$$\alpha 1 = 5.313$$

$$\alpha 2 := 0.03\cdot\rho_s\cdot\frac{f_{yt}\cdot 1000}{\mu_{\mathcal{D}}} \qquad \alpha 2 = 5.313$$

$Pleft = 81.5$ **kips** $Ag = 1018$ sq in

$$vc_L := \min\left[3.5\cdot\sqrt{Fccol\cdot 1000}, \left[\alpha 1\cdot\left(1 + Pleft\cdot\frac{1000}{2000\cdot Ag}\right)\right]\cdot\sqrt{Fccol\cdot 1000}\right] \qquad vc_L = 219 \text{ psi}$$

$$vc_{Cent} := \min\left[3.5\cdot\sqrt{Fccol\cdot 1000}, \left[\alpha 1\cdot\left(1 + P\cdot\frac{1000}{2000\cdot Ag}\right)\right]\cdot\sqrt{Fccol\cdot 1000}\right] \qquad vc_{Cent} = 219 \text{ psi}$$

$$vc_R := \min\left[3.5\cdot\sqrt{Fccol\cdot 1000}, \left[\alpha 1\cdot\left(1 + Prt\cdot\frac{1000}{2000\cdot Ag}\right)\right]\cdot\sqrt{Fccol\cdot 1000}\right] \qquad vc_R = 219 \text{ psi}$$

***If net axial load is less < 0 then Vc = 0***

$$V_{cLT} := \begin{cases} 0 & \text{if } Pleft < 0 \\ vc_L\cdot\frac{Ae}{1000} & \text{otherwise} \end{cases} \qquad V_{cLT} = 178 \text{ kips}$$

$$V_{cCent} := \begin{cases} 0 & \text{if } P < 0 \\ vc_{Cent}\cdot\frac{Ae}{1000} & \text{otherwise} \end{cases} \qquad V_{cCent} = 178 \text{ kips}$$

$$Vc_{RT} := \begin{cases} 0 & \text{if } \mathbf{Prt} < 0 \\ vc_R \cdot \dfrac{\mathbf{Ae}}{1000} & \text{otherwise} \end{cases}$$ $Vc_{RT} = 178$ **kips**

Note : Conserv. use Inside plastic Hinge zone Vc capacity for Outside plastic hinge zone

ART 8.6.2
PAGE 8-10

USE #4 @ 3" SPIRAL

3. **Calculate** $\mathbf{V_s}$ for the transverse reinforcement $f_{yh} := 60$ ksi for spiral

$$\mathbf{Vsmax} := 8 \cdot \mathbf{Ae} \cdot \frac{\sqrt{\mathbf{Fccol} \cdot 1000}}{1000}$$ $\mathbf{Vsmax} = 407$ **kips**

$$\mathbf{V_s} := \mathbf{min}\left[\mathbf{Vsmax}, \left[\frac{\left(\frac{\pi}{2}\right) \cdot \mathbf{A_{bt}} \cdot \mathbf{f_{yh}} \cdot (\mathbf{D} \cdot 12 - 2 \cdot \mathbf{c})}{\mathbf{s}}\right]\right]$$ $\mathbf{V_s} = 203$ **kips**

$\phi := 0.85$

**Conserv. assumed concrete shear strength** = 0 use multiplier, $\Phi 1 := 0$

| | | |
|---|---|---|
| $V_{nLT} := \phi \cdot (V_{cLT} \cdot \Phi 1 + V_s)$ | $V_{nLT} = 173$ **kips** | $V_{pLeft} = 37$ **kips** |
| $V_{nCent} := \phi \cdot (V_{cCent} \cdot \Phi 1 + V_s)$ | $V_{nCent} = 173$ **kips** | $V_{pCenter} = 41$ **kips** |
| $V_{nRT} := \phi \cdot (Vc_{RT} \cdot \Phi 1 + V_s)$ | $V_{nRT} = 173$ **kips** | $V_{pRight} = 44$ **kips** |

**Check D/C ratio** = Vp / Vn

$$\mathbf{LTRatio} := \frac{V_{pLeft}}{V_{nLT}}$$ $\mathbf{LTRatio} = 0.22$

$$\mathbf{CentRatio} := \frac{V_{pCenter}}{V_{nCent}}$$ $\mathbf{CentRatio} = 0.24$

$$\mathbf{RTRatio} := \frac{V_{pRight}}{V_{nRT}}$$ $\mathbf{RTRatio} = 0.3$

**Shear Demand to Capacity Ratio**

If shear D/C ratio < 1 then the frame displacement capacity is governed by flexural deformation.

If **shear D/C ratio > 1** then the frame displacement capacity should be revised to reflect the fact that shear is governing. **Displacement Capacity shall be revised.**

$$\text{Shear D / C } \textbf{Ratio} := \begin{vmatrix} \text{" O.K. "} & \textbf{if } \textbf{LTRatio} < 1 \wedge \textbf{RTRatio} < 1 \wedge \textbf{CentRatio} < 1 \\ \text{" N.G. "} & \textbf{otherwise} \end{vmatrix}$$

$$\text{Shear D / C } \textbf{Ratio} = \text{" O.K. "}$$

If Shear **D / C Ratio < 1** then ***O.K.*** otherwise Displacement Capacity shall be revised.

**Check Displacement Ductility Demand μRatio**

$\mu_D = 2.7$ $\quad \mu_{D\,allowed} = 8$

If $\mu_D$Ratio **<=** $\mu_{D\,allowed}$ then ***O.K.*** otherwise N.G. $\quad \mu_D\textbf{Ratio} = \text{" O.K. "}$

**Displacement Demand - Capacity Ratio**

Establish Demand to Capacity Ratio, R = Δdemand / Δcapacity

$$\mathbf{R2} := \frac{\Delta_d}{\Delta_u} \qquad \boxed{\mathbf{R2} = 0.6}$$

$$\textbf{Rratio2} := \begin{vmatrix} \text{" O.K. "} & \textbf{if } \mathbf{R2} \le 1 \\ \text{" N.G. "} & \textbf{otherwise} \end{vmatrix}$$

$$\textbf{Rratio2} = \text{" O.K. "}$$

If **R2 < 1** then ***O.K.*** otherwise Displacement Capacity shall be revised.

## CAPP Program

**Consider Column Hinged (Pinned) at bottom**

The pinned condition assumption is based on the belief that in the event of a maximum credible earthquake, the column-footing connection would quickly degenerate (degrade) and behave like a pinned condition.

**Run CAPP** program for P-Delta effect case. Open project manager, click on (+) Load Cases, click on (+) Combo1, click on analysis report & read node no.XX from (Horiz axis title) displacement curve then close analysis report. Click on node no.XX. Once Node information window open up then click on red color symbol (bottom right corner of window) to view plot on screen. From this plot read values for red color & green color points. Perform same procedure for without P-Delta effect case. Plot total X Push load vs. X Displacement for P-Delta and without P-Delta case on the same plot. Read Push load vs. X displacement value for a point where two curve intersect OR read values for First Yield push load & First Yield displacement.

From plot X Push load = **xxxx** kips & X Displacement = **xxxx** ft at 1st Yield displ & Push load

$$X_{pushload} := 79.0\ \text{kips} \qquad X_{Disp} := 0.3421\ \text{Ft}$$

Initial effect. stiffness. $\quad Keff := \dfrac{X_{pushload}}{X_{Disp} \cdot 12} \qquad Keff = 19\ \dfrac{kips}{in}$

**Calculate an approximate Fundamental Period, Tf**

$$Tf := 0.32 \cdot \sqrt{\frac{WDL}{Keff}} \qquad Tf = 1.9\ \text{sec}$$

**Determine the Damped Elastic Acceleration Response Spectrum (ARS) at the site in g's.**

By using the given site spectrum and above calculated period, the corresponding ARS for 5% damping.

$ARS := 0.45$ Read from Acceleration Response Spectrum curve

**Calculate the Displacement Demand Dd**

$$Dd := ARS \cdot \frac{WDL}{Keff} \qquad Dd = 15.9\ \text{in}$$

**Displacement Capacity** $\quad \Delta_{ult} := 24.1$ in from CAPP output "X Ultimate Displ."

$$\Delta Ratio := \frac{Dd}{\Delta_{ult}} \qquad \Delta Ratio = 0.7$$

$$\frac{Dd}{\Delta_{ult}} \quad Ratio3 := \begin{cases} \text{"O.K."} & \text{if } \Delta Ratio < 1 \\ \text{"N.G."} & \text{otherwise} \end{cases}$$

$$\frac{Dd}{\Delta_{ult}} \quad Ratio3 = \text{"O.K."}$$

If $\dfrac{Dd}{\Delta_{ult}}$ **Ratio3 < 1** then *O.K.* otherwise Displacement Capacity shall be revised.

EXAMPLE1 (MO) SP

34

# CAPP Project Report

Loading Name: Combo1

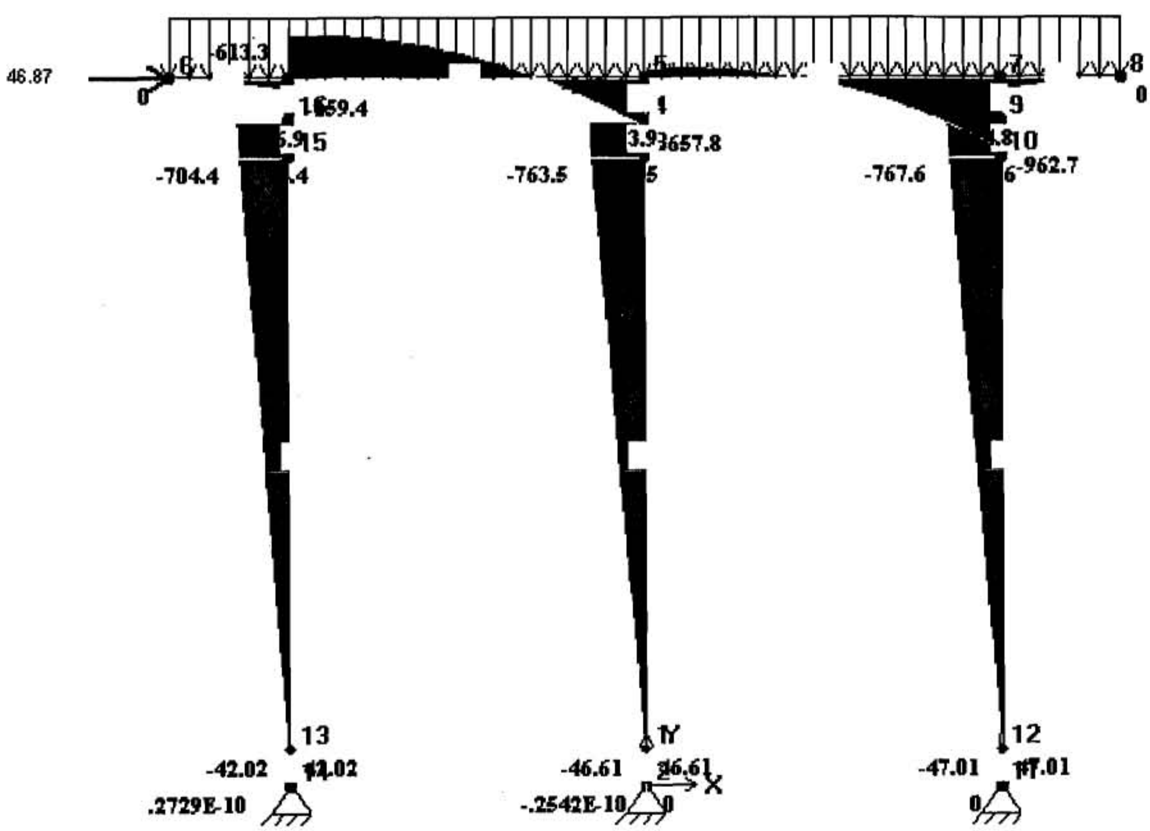
3/15/2006

Report Type: Moment Diagram - Units: kips-ft

Comments:

Page __ of __

$m = -963\ k\text{-}'$



## Beam cap Design

Width, $B := 42$ in $H := 48$ in

$Fc := 3$ ksi $Fy := 60$ ksi

Moment from CAPP ouput $M_y := 963$ k − ft

$f_r := 0.37 \cdot \left[\left(\frac{Fc}{1}\right)^{0.5}\right]$ $f_r = 0.641$ ksi LRFD 5.4.2.6

$Ig := \frac{1}{12} \cdot B \cdot H^3$ $Ig = 387072$ $in^4$ $Y_t := \frac{H}{2}$

$S_c := \frac{Ig}{Y_t}$ $S_c = 16128$ $in^3$

$Mcr := S_c \cdot \frac{f_r}{12}$ $Mcr = 861$ k − ft LRFD 5.7.3..3.2

$1.2Mcr = 1034$ k − ft

**Long dir bar**

$1.33 \cdot M_y = 1281$ k − ft

Use $M_{ydesign} := \min(1.2Mcr, 1.33M_y)$ $M_{ydesign} = 1034$ k − ft Rebar Cover $C := 1.5$ in

Stirrups, $S_{bar} := 5$ Side cover $C1 := 1.5$ in

Long bar # $Lbar := 8$ Long bar dia $Db_L := \frac{Lbar}{8}$ $As_{Lbar} := \frac{\pi}{4} \cdot Db_L^2$ $As_{Lbar} = 0.785\ in^2$

$\phi := 0.9$ $dc_L := H - C - \frac{S_{bar}}{8} - \frac{Db_L}{2}$ $dc_L = 45.38$ in LRFD 5.5.4.2

$Ru_y := M_{ydesign} \cdot \frac{12}{\phi \cdot B \cdot dc_L^2}$ $Ru_y = 0.159$

$$Rw_y := \frac{0.85 \cdot Fc}{Fy} \cdot \left[1 - \left(1 - 2\frac{Ru_y}{0.85Fc}\right)^{0.5}\right] \qquad Rw_y = 0.0027$$

As req'd per ft = $As_l := Rw_y \cdot B \cdot dc_L$ $\quad As_l = 5.231 \quad in^2$

No. of long bar req'd, $Nb_L := \frac{As_l}{As_{Lbar}}$ $\quad Nb_L = 7$

$UsedNb_L := 8$

$\boxed{\text{Use 8 - \# 8}}$ @ Top & Bottom in long dir.

Barspacing, $S := \dfrac{B - 2 \cdot \frac{S_{bar}}{8} - C1 - C}{UsedNb_L - 1}$

$S = 5.39 \quad in$

**Shrinkage and temperature reinf.** LRFD 5.10.8.2

$Ag := H \cdot B$

$0.11 \cdot \frac{Ag}{Fy} = 3.696$ $\qquad$ $0.0015 \cdot Ag = 3.024$

$As_{shr} := \min\left[\left(\frac{0.11 \cdot Ag}{Fy}\right), 0.0015 \cdot Ag\right]$

$As_{shr} = 3.024$

$As_{top} := \frac{As_{shr}}{2}$

Required As for shrinkage @ top & bottom in either dir $As_{top} = 1.512$ $in^2$

$$ShrinkageReinf := \begin{vmatrix} \text{" O. K. "} & \text{if } As_{top} < As_l \\ \text{" N. G. "} & \text{otherwise} \end{vmatrix}$$

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

**Crack Control :**

$\gamma_e := 1$ Assume class 2 explosure LRFD 5.7.3.4

$Es := 29000$ $Ec := 3300$

$$d_c := C + \frac{S_{bar}}{8} + \frac{\frac{Lbar}{8}}{2} \qquad d_c = 2.625$$

$$\beta_s := 1 + \frac{d_c}{0.7 \cdot (H - d_c)}$$

$$n := \mathrm{round}\left(\frac{Es}{Ec}\right) \qquad n = 9 \qquad \beta_s = 1.083$$

$$\rho := \frac{As_l}{B \cdot dc_L}$$

$$k := \left[(\rho \cdot .n)^2 + (2 \cdot \rho \cdot n)\right]^{0.5} - \rho \cdot .n \qquad k = 0.222$$

$$j := 1 - \frac{k}{3}$$

$Mu := 700$ k − ft For service limit case

$$f_s := \frac{Mu \cdot 12}{As_l \cdot j \cdot dc_L} \qquad f_s = 38.22 \text{ ksi}$$

$$S_{max} := 700 \cdot \frac{\gamma_e}{\beta_s \cdot f_s} - 2 \cdot d_c$$

$S_{max} = 11.67$ in Rebar spacing used, $S = 5.39$ in

$$BarSpace_{used} := \begin{vmatrix} \text{" O. K. "} & \text{if } S < S_{max} \\ \text{" N. G. "} & \text{otherwise} \end{vmatrix}$$

$$\boxed{BarSpace_{used} = \text{" O. K. "}}$$

**Beam Joint Design** **SDC C or D** **Article 8.13.2**

Col Diam. $Dc := 36$ in

Beam depth $Ds := 36$ in

Beam cap width, $Bf := 42$ in

$fc := 3000$ psi

$fce := \min(1.1 \cdot fc, 5000)$

$fce = 3.3 \times 10^{3}$ psi

$L_{ac} := 30$ inch

$A_{jh} := (Dc + Ds) \cdot Bf$

$A_{jv} := L_{ac} \cdot Bf$

**Left Column**

Pcol = Column axial force including the effects of overturning

$Pcol_L := 81000$ lbs

$MPcol_L := 828100$ lb − ft

$$T_{cL} := 1.2 \cdot \frac{MPcol_L \cdot 12}{0.7 \cdot Dc}$$

$$v_{jvL} := \frac{T_{cL}}{A_{jv}}$$ Eqn 8.42

$v_{jvL} = 376$ psi

$$f_{vL} := \frac{Pcol_L}{A_{jh}}$$ Eqn 8.44

$f_{vL} = 27$ psi

$P_{bL} := 25000$ lbs

$$f_{hL} := \frac{P_{bL}}{Bf \cdot Ds}$$ Eqn. 8.46

**Center Column**

$Pcol_C := 177000$ lbs

$MPcol_C := 909300$ lb − ft

$$T_{cC} := 1.2 \cdot \frac{MPcol_C \cdot 12}{0.7 \cdot Dc}$$

$$v_{jvC} := \frac{T_{cC}}{A_{jv}}$$

$v_{jvC} = 412$ psi

$$f_{vC} := \frac{Pcol_C}{A_{jh}}$$

$f_{vC} = 59$ psi

$P_{bC} := 25000$ lbs

$$f_{hC} := \frac{P_{bC}}{Bf \cdot Ds}$$

**Right Column**

$Pcol_R := 272000$ lbs

$MPcol_R := 982000$ lb − ft

$$T_{cR} := 1.2 \cdot \frac{MPcol_R \cdot 12}{0.7 \cdot Dc}$$

$$v_{jvR} := \frac{T_{cR}}{A_{jv}}$$

$v_{jvR} = 445$ psi

$$f_{vR} := \frac{Pcol_R}{A_{jh}}$$

$f_{vR} = 90$ psi

$P_{bR} := 25000$ lbs

$$f_{hR} := \frac{P_{bR}}{Bf \cdot Ds}$$

$$T_c = \frac{Mo\ Col}{h}$$

$h = 0.7 * Dc$

$Mo\ Col = 1.2 * Mp$

**Principal Tension** Eqn 8.40

$$P_{tL} := \left| \frac{f_{hL} + f_{vL}}{2} - \sqrt{\left(\frac{f_{hL} - f_{vL}}{2}\right)^2 + v_{jvL}^2} \right|$$

$P_{tL} = 354$ psi

$$P_{tR} := \left| \frac{f_{hR} + f_{vR}}{2} - \sqrt{\left(\frac{f_{hR} - f_{vR}}{2}\right)^2 + v_{jvR}^2} \right|$$

$P_{tR} = 394$ psi

$$P_{tC} := \left| \frac{f_{hC} + f_{vC}}{2} - \sqrt{\left(\frac{f_{hC} - f_{vC}}{2}\right)^2 + v_{jvC}^2} \right|$$

$P_{tC} = 375$ psi

**Principal Compression** Eqn 8.41

$$P_{cL} := \frac{f_{hL} + f_{vL}}{2} + \sqrt{\left(\frac{f_{hL} - f_{vL}}{2}\right)^2 + v_{jvL}^2}$$

$P_{cL} = 397$ psi

$$P_{cR} := \frac{f_{hR} + f_{vR}}{2} + \sqrt{\left(\frac{f_{hR} - f_{vR}}{2}\right)^2 + v_{jvR}^2}$$

$P_{cR} = 500$ psi

$$P_{cC} := \frac{f_{hC} + f_{vC}}{2} + \sqrt{\left(\frac{f_{hC} - f_{vC}}{2}\right)^2 + v_{jvC}^2}$$

$P_{cC} = 450$ psi

$P_{cmax} := 0.25 \cdot fce$ $P_{cmax} = 825$ Eqn 8.38

$P_{tmax} := 12 \cdot \sqrt{fce}$ $P_{tmax} = 689$ Eqn 8.39

$$\text{PrincipalCompression} := \begin{array}{|l} \text{" O.K. " if } P_{cL} \le P_{cmax} \wedge P_{cC} \le P_{cmax} \wedge P_{cR} \le P_{cmax} \\ \text{" N.G. " otherwise} \end{array}$$

$$\text{PrincipalTension} := \begin{array}{|l} \text{" O.K. " if } P_{tL} \le P_{tmax} \wedge P_{tC} \le P_{tmax} \wedge P_{tR} \le P_{tmax} \\ \text{" N.G. " otherwise} \end{array}$$

PrincipalCompression = " O.K. " PrincipalTension = " O.K. "

**Note** :

If Principal Compression OR Principal Tension is Not "O.K." then Beam properties (thickness or/and width) or/and Column Diameter shall be increased.

**Provide Min Hoop bars in the beam cap for SDC C**

$f_{yh} := 60000$ psi $\qquad \rho_{smin} := \dfrac{3.5}{f_{yh}} \cdot \sqrt{fce}$ $\qquad \rho_{smin} = 0.0034$ $\qquad$ Eqn 8.47

**Hoop bar** information

#4 @ 3"

$A_{bt} := 0.2$ sq. in $\quad s := 3$ in $\quad c := 1.5$ inch conc. cover for long steel

$$\rho_s := 4 \cdot A_{bt} \cdot \frac{1}{(Dc - 2 \cdot c) \cdot s} \qquad \rho_s = 0.0081$$

$$\rho_{s1} := \begin{array}{|l} \text{" O.K. "} \ \text{if } \rho_{smin} < \rho_s \\ \text{" N.G. "} \ \text{otherwise} \end{array}$$

Hoop Bar Reinf $\rho_{s1} = $ " O.K. "

**Check Hoop bar or Add'l steel req't for SDC D**

$P_{tL} = 354$ psi $\qquad P_{tC} = 375$ psi $\qquad P_{tR} = 394$ psi

$P_{tvalue} := 3.5 \cdot \sqrt{fce}$ $\qquad P_{tvalue} = 201$ psi

$$PrincipalTension := \begin{array}{|l} \text{"O.K. so provide min Hoop bar from above "} \ \text{if } P_{tL} \le P_{tvalue} \wedge P_{tC} \le P_{tvalue} \wedge P_{tR} \le P_{tvalue} \\ \text{" N.G. "} \ \text{otherwise} \end{array}$$

PrincipalTension = " N.G. "

Note

If Principal Tension is **N.G.** then provide add'l reinf. in the beam cap for **SDC D ONLY** $\qquad$ article 8.13.4.3

Column long steel, $A_{st} := 10.21 \ in^2$

**A ) Vertical Stirrups:**

$A_{sjv} := 0.2 \cdot A_{st}$ $A_{sjv} = 2.042$ $in^2$ Each side of column within a column diameter distance from center of column (1/2 column diameter from face of column)

**Check near center column**

Vertical stirrups for other load cases provided in this area $A_s := 0.305$ # 5 @ 12"

$A_{sprovided} := 2 \cdot A_s \cdot \frac{Dc}{2} \cdot \frac{1}{St}$ $St := 12$ inch $A_{sprovided} = 0.915$ for other load

Additional Steel required, $Asjv := A_{sjv} - A_{sprovided}$

$Asjv = 1.127$ $in^2$ in $\frac{1}{2} Dc = 18$ inch

Additional No. of bar in 1/2 Dc $Nb_{single} := 1$ space single stirrups between other vertical stirrups

$Nb_{as} := Nb_{single} \cdot A_s \cdot 2$ $Nb_{as} = 0.61$

$$\text{Vertstirrupswithsingle} := \begin{cases} \text{"O. K. "} & \text{if } Asjv \le Nb_{as} \\ \text{" N. G. "} & \text{otherwise} \end{cases}$$

$\boxed{\text{Vertstirrupswithsingle} = \text{" N. G. "}}$

**Note: Try Double stirrup only if "single stirrups" combination** Vertstirrupswithsingle **= "N.G."**

Additional No. of bar in 1/2 Dc $Nb_{Double} := 1$ space Double stirrups between other vertical stirrups

$Nb_{Das} := Nb_{Double} \cdot A_s \cdot 4$ $Nb_{Das} = 1.22$

$$\text{VertstirrupswithDouble} := \begin{cases} \text{"O. K. "} & \text{if } Asjv \le Nb_{Das} \\ \text{" N. G. "} & \text{otherwise} \end{cases}$$

$Asjv = 1.127$

$\text{VertstirrupswithDouble} = \text{"O. K. "}$

**Check near exterior column**

Vertical stirrups for other load cases provided in this area $A_{s1} := 0.6$ # 5 @ 6" (DBL)

$A_{sprovidedEx} := 2 \cdot A_{s1} \cdot \frac{Dc}{2} \cdot \frac{1}{St1}$ $St1 := 6$ $A_{sprovidedEx} = 3.6$

Additional Steel required, $Asjv_{ex} := A_{sjv} - A_{sprovidedEx}$

$Asjv_{ex} = -1.558\ in^2$ in $\frac{1}{2} Dc = 18$

$Asjv = 1.127$

Additional No. of bar in 1/2 Dc

$$AddVertstirrups_{ex} := \begin{cases} \text{"Add'l steel not req'd"} & \text{if } Asjv \le A_{sprovidedEx} \\ \text{"Add'l steel req'd"} & \text{otherwise} \end{cases}$$

Near exterior column $\boxed{AddVertstirrups_{ex} = \text{"Add'l steel not req'd"}}$

**Note :**

**B ) Horiz stirrups:**

Assumed Horiz stirrups Not req'd for this type of bridge

**C ) Horiz Side Reinf:**

$Atop_{long} := 6.28\ in^2$ $\qquad Abot_{long} := 6.28\ in^2$

$As_{eachface} := 0.1 \cdot \max(Atop_{long}, Abot_{long})$ $\qquad As_{eachface} = 0.628$ EQN. 8.50

Side reinf provided **4-#6 @** each face $\quad As_{side} := 1.76$

$$Sidereinf := \begin{cases} \text{"O. K."} & \text{if } As_{eachface} \le As_{side} \\ \text{"N. G."} & \text{otherwise} \end{cases} \qquad Sidereinf = \text{"O. K."}$$

**D ) J - Dowels:** If skew > 20 deg then provide add'l reinf as per eqn. ~~8.52~~ 8.51

skew = 0 **No J - Dowel Req'd**

**E ) Transverse Reinforcement:**

Anchorage length of column stee $l_{ac} := 34$ inch

$$\rho_{st} := 0.4 \cdot \frac{A_{st}}{l_{ac}^{\ 2}}$$ EQN 8.52

$\rho_{st} = 0.0035$

$\rho_s = 0.0081$ **For Hoop bar from calc above**

$$Hoopbar := \begin{cases} \text{"O. K."} & \text{if } \rho_{st} \le \rho_s \\ \text{"N. G."} & \text{otherwise} \end{cases}$$

$Hoopbar = \text{"O. K."}$ for Transverse Reinf. req't

**Note** :

If Hoop bar is "N. G." then hoop bar size or/and spacing shall be revised in the above calc.

**Footing Joint Shear** SDC C and D **Article 6.4.5**

Col Diam. $D := 36$ in

$fc := 3000$ psi

Footing depth $Df := 48$ in

$fce := \min(1.1 \cdot fc, 5000)$

$B_{eff} := D \cdot \sqrt{2}$

$fce = 3.3 \times 10^3$ psi

$A_{jh} := (D + Df)^2$

**Left Column** **Center Column** **Right Column**

Pcol = Column axial force including the effects of overturning

$Pcol_L := 81000$ lbs

$Pcol_C := 17700$ lbs

$Pcol_R := 272000$ lbs

Tjv = net tensile force @ Footing or Pile cap

Tjv = 0, If No uplift or uplift force < Friction capacity

$T_{jvL} := 0$ lbs

$T_{jvC} := 0$ lbs

$T_{jvR} := 0$ lbs

$$v_{jvL} := \frac{T_{jvL}}{B_{eff} \cdot Df}$$

$$v_{jvC} := \frac{T_{jvC}}{B_{eff} \cdot Df}$$

$$v_{jvR} := \frac{T_{jvR}}{B_{eff} \cdot Df}$$

$v_{jvL} = 0$

$v_{jvC} = 0$

$v_{jvR} = 0$

$$f_{vL} := \frac{Pcol_L}{A_{jh}}$$

$$f_{vC} := \frac{Pcol_C}{A_{jh}}$$

$$f_{vR} := \frac{Pcol_R}{A_{jh}}$$

$f_{vL} = 11$ psi

$f_{vC} = 25$ psi

$f_{vR} = 39$ psi

**Principal Tension**

$$P_{tL} := \left| \frac{f_{vL}}{2} - \sqrt{\left(\frac{f_{vL}}{2}\right)^2 + v_{jvL}^2} \right|$$

$P_{tL} = 0$ psi

$$P_{tC} := \left| \frac{f_{vC}}{2} - \sqrt{\left(\frac{f_{vC}}{2}\right)^2 + v_{jvC}^2} \right|$$

$P_{tC} = 0$ psi

$$P_{tR} := \left| \frac{f_{vR}}{2} - \sqrt{\left(\frac{f_{vR}}{2}\right)^2 + v_{jvR}^2} \right|$$

$P_{tR} = 0$ psi

**Principal Compression**

$$P_{cL} := \frac{f_{vL}}{2} + \sqrt{\left(\frac{f_{vL}}{2}\right)^2 + v_{jvL}^{\,2}} \qquad P_{cC} := \frac{f_{vC}}{2} + \sqrt{\left(\frac{f_{vC}}{2}\right)^2 + v_{jvC}^{\,2}} \qquad P_{cR} := \frac{f_{vR}}{2} + \sqrt{\left(\frac{f_{vR}}{2}\right)^2 + v_{jvR}^{\,2}}$$

$P_{cL} = 11$ psi $\qquad P_{cC} = 25$ psi $\qquad P_{cR} = 39$ psi

$P_{cmax} := 0.25 \cdot fce \qquad P_{cmax} = 825$ Eqn 6.10

$P_{tmax} := 12 \cdot \sqrt{fce} \qquad P_{tmax} = 689$ Eqn 6.11

$$\text{PrincipalCompression} := \begin{cases} \text{" O.K. "} & \text{if } P_{cL} \le P_{cmax} \wedge P_{cC} \le P_{cmax} \wedge P_{cR} \le P_{cmax} \\ \text{" N.G. "} & \text{otherwise} \end{cases}$$

$$\text{PrincipalTension} := \begin{cases} \text{" O.K. "} & \text{if } P_{tL} \le 12 \cdot \sqrt{fce} \wedge P_{tC} \le 12 \cdot \sqrt{fce} \wedge P_{tR} \le 12 \cdot \sqrt{fce} \\ \text{" N.G. "} & \text{otherwise} \end{cases}$$

PrincipalCompression = " O.K. "

PrincipalTension = " O.K. "

**Note** :

If Principal Compression OR Principal Tension is Not "O.K." then Footing thickness shall be increased.

**PILE FOOTING DESIGN**
**(AASHTO 4.4)**

SP

File:Pseicft21bt23

Date: 08/24/06

Bridge **EXAMPLE1** (MO)@ County **Pemiscot** **Column #** **1**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Bent # | 2 | | | | | | |
| # of columns = | 3 | | *Calculate FWS reduction factor:* | | Footing volume (ft.^3) = | 676 | Footing weight (kips) = 101.40 |
| Column diameter (ft.) = | 3 | | roadway width (ft.) = | 38.83333 | Add. column Ht. (ft.)* = | 0 | Fill weight = 58.30 |
| Fill depth (ft.) = | 3 | | 1/2 of adjacent spans (ft.) = | 60 | | | Add. column weight = 0.00 |
| **Number of piles** =** | 9 | 14" CIP | FWS reduction (k/col.) = | 27.18 | | | Max add. (ftg+col+fill) = 159.70 |
| Footing depth (ft.) = | 4 | | | | | | Min Add. (ftg+col-FWS) = 74.22 |
| Ft length along Z axis (ft.) = | 13 | *OR* | Mini. Ftg length (ft.)*** = | | | | Min eccentricity (ft), e' = 0.14 |
| Ft width along Y axis (ft) = | 13 | *OR* | Mini. Ftg width (ft.)*** = | | **W/Seismic** **Max** | **56.72** Ton/pile | |
| | | | | | **W/Seismic** **Min** | **-19.01** Ton/pile | |

Group sect. mod., Sz = 30
Group sect. mod., Sy = 30

30
30

Lenght to widht ratio = 1.00

| AASHTO LRFD Table 3.4.1-1 |
|---|
| Load Combination & Load factors |

Note: Sz and Sy are group section modulus based on spacing ratio, and equal to sum of centroid dist. times # of piles divided by centroid dist.
where Sz is about bridge direction.

| **Recommended LRFD Guidelines section 6.4.2 & 8.5** | for seismic design |
|---|---|
| Pile size allowed 16" Used 14" CIP | |
| Cant'r dist. From face of column to edge of pile cap, Lftg = 5 | |
| Ratio of Lftg/Ftg depth = 1.25 <= 2.5 | If Ratio of Lftg/Ftg depth >2.5 then use modification factor = 1.2 |
| Factor 1 | |
| Mp(o) = Mp * Factor | |

Seismic Load comb

| | Long dir. | Trans dir. |
|---|---|---|
| Factor | 1 | 1 |

**Group 7 = 1.00 (Earth quake)**

where,
e (y) is along footing lenght
e (z) is along footing width

Pmax = Axial + (Max. add) / load factor
Pmin = Axial + (Min. add) / load factor
Me = Pmax * e'

| Mp(o) | Load Group | Load Factor | Axial | My | Mz | Pmax | Me | Four possible load cases (Tons) Pmax (1) | Pmax (2) | Pmax (3) | Pmax (4) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | **SEISMIC** | | | | | | | | | | |
| 0 | Column1 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 |
| 910 | Column1 | 1.0 | 177 | 910 | 910 | 336.70 | 0.00 | 49.04 | 18.71 | 33.87 | 33.87 |
| 0 | Column2 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 |
| 829 | Column2 | 1.0 | 81 | 829 | 829 | 240.70 | 0.00 | 41.01 | 13.37 | 27.19 | 27.19 |
| 0 | Column3 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 |
| 982 | Column3 | 1.0 | 272 | 982 | 982 | 431.70 | 0.00 | 56.72 | 23.98 | 40.35 | 40.35 |
| | **for seismic analysis.** | | | | 56.72 | SEISMIC | MAXIMUM | 56.72 | 23.98 | 40.35 | 40.35 |
| | | | | | -19.01 | SEISMIC | MINIMUM | 8.87 | 8.87 | 8.87 | 8.87 |

| Load Group | Pmin | M'e | Four possible load cases (Tons) Pmin (1) | Pmin (2) | Pmin (3) | Pmin (4) | TENSION PER PILE (TONS) | Axial | Mp | Mp(o) |
|---|---|---|---|---|---|---|---|---|---|---|
| **SEISMIC** | | | | | | | | | | |
| Column1 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| Column1 | 251.22 | 0.00 | -16.38 | 13.96 | -1.21 | -1.21 | -16.38 | 177 | 910 | 910 |
| Column2 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| Column2 | 155.22 | 0.00 | -19.01 | 8.62 | -5.19 | -5.19 | -19.01 | 81 | 829 | 829 |
| Column3 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| Column3 | 346.22 | 0.00 | -13.50 | 19.23 | 2.87 | 2.87 | -13.50 | 272 | 982 | 982 |
| for seismic analysis. (MAXIMUM) | | | 4.12 | 19.23 | 4.12 | 4.12 | | | | |
| (MINIMUM) | | | -19.01 | 4.12 | -5.19 | -5.19 | | | | |

| | |
|---|---|
| **Maximum Design Pile Load (tons/pile) =** | **56.72** |
| **Tension (tons/pile) =** | **-19.01** |
| ***Pile Dimension :*** | 9 14" CIP |
| ***Footing Dimension :*** | **13.00' L x 13.00' W x 4.00' D** |

tension

Pmax (1) = Pmax/# of Pile + My/Sy + Mz/Sz
Pmax (2) = Pmax /# of Pile + Me/min(Sz,Sy)
Pmax (3) = Pmax/# of Pile + Me/Sy + Mz/Sz
Pmax (4) = Pmax/# of Pile + My/Sy + Me/Sz

Pmin (1) = Pmin/# of Pile - My/Sy - Mz/Sz
Pmin (2) = Pmin /# of Pile - Me/min(Sz,Sy)
Pmin (3) = Pmin/# of Pile - Me/Sy - Mz/Sz
Pmin (4) = Pmin/# of Pile - My/Sy - Me/Sz

CONSER. USE THIS LOAD CASE FOR PILE CAP DESIGN,
P = 57 TON/PILE < PILE ULTIMATE CAPACITY
= 114 K/PILE

47

**PILE FOOTING DESIGN**
**(AASHTO 4.4)**

SP

File:Pseicft21bt23

Date: 08/24/06

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Bridge | EXAMPLE1 (MO) | County | Pemiscot | Column # | 1 | | |
| Bent # | 2 | | | | | | |
| # of columns = | 3 | *Calculate FWS reduction factor:* | | Footing volume (ft.^3) = | 676 | Footing weight (kips) = | 101.40 |
| Column diameter (ft.) = | 3 | roadway width (ft.) = | 38.83333 | Add. column Ht. (ft.)* = | 0 | Fill weight = | 58.30 |
| Fill depth (ft.) = | 3 | 1/2 of adjacent spans (ft.) = | 60 | | | Add. column weight = | 0.00 |
| **Number of piles** =** | 9 | 14" CIP / FWS reduction (k/col.) = | 27.18 | | | Max add. (ftg+col+fill) = | 159.70 |
| Footing depth (ft.) = | 4 | | | | | Min Add. (ftg+col-FWS) = | 74.22 |
| Ft length along Z axis (ft.) = | 13 | *OR* Mini. Ftg length (ft.)*** = | | | | Min eccentricity (ft), e' = | 0.14 |
| Ft width along Y axis (ft) = | 13 | *OR* Mini. Ftg width (ft.)*** = | | **W/Seismic** **Max** | 40.35 Ton/pile | | |
| | | | | **W/Seismic** **Min** | -5.19 Ton/pile | | |

Group sect. mod., Sz = 30
Group sect. mod., Sy = 30

30
30

Lenght to widht ratio = 1.00

AASHTO LRFD Table 3.4.1-1
Load Combination & Load factors

Note: Sz and Sy are group section modulus based on spacing ratio, and equal to sum of centroid dist. times # of piles divided by centroid dist.
where Sz is about bridge direction.

**Recommended LRFD Guidelines section 6.4.2 & 8.5** for seismic design
Plie size allowed 16" Used 14" CIP
Cant'r dist. From face of column to edge of pile cap, Lftg = 5
Ratio of Lftg/Ftg depth = 1.25 <= 2.5 If Ratio of Lftg/Ftg depth >2.5 then use modification factor = 1.2
Factor 1
Mp(o) = Mp * Factor

Seismic Load comb

| | Long dir. | Trans dir. |
|---|---|---|
| Factor | 0 | 1 |

**Group 7 =** **1.00** **(Earth quake)**

where,
e (y) is along footing lenght
e (z) is along footing width

Pmax = Axial + (Max. add) / load factor
Pmin = Axial + (Min. add) / load factor
Me = Pmax * e'

| Mp(o) | Load Group | Load Factor | Axial | My | Mz | Pmax | Me | Four possible load cases (Tons) Pmax (1) | Pmax (2) | Pmax (3) | Pmax (4) | Pmin | M'e | Four possible load cases (Tons) Pmin (1) | Pmin (2) | Pmin (3) | Pmin (4) | TENSION PER PILE (TONS) | Axial | Mp | Mp(o) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **SEISMIC** | | | | | | | | | | | | | | | | | | | | |
| 0 | Column1 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| 910 | Column1 | 1.0 | 177 | 0 | 910 | 336.70 | 0.00 | 33.87 | 18.71 | 33.87 | 18.71 | 251.22 | 0.00 | -1.21 | 13.96 | -1.21 | 13.96 | -1.21 | 177 | 910 | 910 |
| 0 | Column2 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| 829 | Column2 | 1.0 | 81 | 0 | 829 | 240.70 | 0.00 | 27.19 | 13.37 | 27.19 | 13.37 | 155.22 | 0.00 | -5.19 | 8.62 | -5.19 | 8.62 | -5.19 | 81 | 829 | 829 |
| 0 | Column3 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| 982 | Column3 | 1.0 | 272 | 0 | 982 | 431.70 | 0.00 | 40.35 | 23.98 | 40.35 | 23.98 | 346.22 | 0.00 | 2.87 | 19.23 | 2.87 | 19.23 | | 272 | 982 | 982 |
| | **for seismic analysis.** | | | | 40.35 | SEISMIC | MAXIMUM | 40.35 | 23.98 | 40.35 | 23.98 | | | 4.12 | 19.23 | 4.12 | 19.23 | | | | |
| | | | | | -5.19 | SEISMIC | MINIMUM | 8.87 | 8.87 | 8.87 | 8.87 | | | -5.19 | 4.12 | -5.19 | 4.12 | | | | |

**Maximum Design Pile Load (tons/pile) = 40.35**
**Tension (tons/pile) = -5.19**
***Plie Dimension :*** **9 14" CIP**
***Footing Dimension :*** **13.00' L x 13.00' W x 4.00' D**

tension

Pmax (1) = Pmax/# of Pile + My/Sy + Mz/Sz
Pmax (2) = Pmax /# of Pile + Me/min(Sz,Sy)
Pmax (3) = Pmax/# of Pile + Me/Sy + Mz/Sz
Pmax (4) = Pmax/# of Pile + My/Sy + Me/Sz

Pmin (1) = Pmin/# of Pile - My/Sy - Mz/Sz
Pmin (2) = Pmin /# of Pile - Me/min(Sz,Sy)
Pmin (3) = Pmin/# of Pile - Me/Sy - Mz/Sz
Pmin (4) = Pmin/# of Pile - My/Sy - Me/Sz

# PILE FOOTING DESIGN

(AASHTO 4.4) SP File:Pseicft21bt23 Date: 08/24/06

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Bridge | EXAMPLE1 (MO) | County | Pemiscot | Column # | 1 | | |
| Bent # | 2 | | | | | | |
| # of columns = | 3 | *Calculate FWS reduction factor:* | | Footing volume (ft.^3) = | 676 | Footing weight (kips) = | 101.40 |
| Column diameter (ft.) = | 3 | roadway width (ft.) = | 38.83333 | Add. column Ht. (ft.)* = | 0 | Fill weight = | 58.30 |
| Fill depth (ft.) = | 3 | 1/2 of adjacent spans (ft.) = | 60 | | | Add. column weight = | 0.00 |
| **Number of piles** =** | 9 | 14" CIP | FWS reduction (k/col.) = 27.18 | | | Max add. (ftg+col+fill) = | 159.70 |
| Footing depth (ft.) = | 4 | | | | | Min Add. (ftg+col-FWS) = | 74.22 |
| Ft length along Z axis (ft.) = | 13 | *OR* | Mini. Ftg length (ft.)*** = | | | Min eccentricity (ft), e' = | 0.14 |
| Ft width along Y axis (ft) = | 13 | *OR* | Mini. Ftg width (ft.)*** = | **W/Seismic** | **Max** | 48.53 | Ton/pile |
| | | | | **W/Seismic** | **Min** | -12.10 | Ton/pile |

Group sect. mod., Sz = 30
Group sect. mod., Sy = 30

30
30

Lenght to widht ratio = 1.00

| AASHTO LRFD Table 3.4.1-1 |
|---|
| Load Combination & Load factors |

Note: Sz and Sy are group section modulus based on spacing ratio, and equal to sum of centroid dist. times # of piles divided by centroid dist.
where Sz is about bridge direction.

| **Recommended LRFD Guidelines section 6.4.2 & 8.5** for seismic design | | | | |
|---|---|---|---|---|
| Plie size allowed | 16" | Used | 14" CIP | |
| Cant'r dist. From face of column to edge of pile cap, Lftg = | 5 | | | |
| Ratio of Lftg/Ftg depth = | 1.25 | <= 2.5 | If Ratio of Lftg/Ftg depth >2.5 then use modification factor = 1.2 | |
| Factor | 1 | | | |
| Mp(o) = | Mp * Factor | | | |

Seismic Load comb

| | Long dir. | Trans dir. |
|---|---|---|
| Factor | 0.5 | 1 |

Group 7 = 1.00 (Earth quake)

where,
e (y) is along footing lenght
e (z) is along footing width

Pmax = Axial + (Max. add) / load factor
Pmin = Axial + (Min. add) / load factor
Me = Pmax * e'

| Mp(o) | Load Group | Load Factor | Axial | My | Mz | Pmax | Me | Four possible load cases (Tons) Pmax (1) | Pmax (2) | Pmax (3) | Pmax (4) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | SEISMIC | | | | | | | | | | |
| 0 | Column1 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 |
| 910 | Column1 | 1.0 | 177 | 455 | 910 | 336.70 | 0.00 | 41.46 | 18.71 | 33.87 | 26.29 |
| 0 | Column2 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 |
| 829 | Column2 | 1.0 | 81 | 414.5 | 829 | 240.70 | 0.00 | 34.10 | 13.37 | 27.19 | 20.28 |
| 0 | Column3 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 |
| 982 | Column3 | 1.0 | 272 | 491 | 982 | 431.70 | 0.00 | 48.53 | 23.98 | 40.35 | 32.17 |
| | for seismic analysis. | | | | 48.53 | SEISMIC | MAXIMUM | 48.53 | 23.98 | 40.35 | 32.17 |
| | | | | | -12.10 | SEISMIC | MINIMUM | 8.87 | 8.87 | 8.87 | 8.87 |

| Load Group | Pmin | M'e | Four possible load cases (Tons) Pmin (1) | Pmin (2) | Pmin (3) | Pmin (4) | TENSION PER PILE (TONS) | Axial | Mp | Mp(o) |
|---|---|---|---|---|---|---|---|---|---|---|
| Column1 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| Column1 | 251.22 | 0.00 | -8.79 | 13.96 | -1.21 | 6.37 | -8.79 | 177 | 910 | 910 |
| Column2 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| Column2 | 155.22 | 0.00 | -12.10 | 8.62 | -5.19 | 1.71 | -12.10 | 81 | 829 | 829 |
| Column3 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| Column3 | 346.22 | 0.00 | -5.32 | 19.23 | 2.87 | 11.05 | -5.32 | 272 | 982 | 982 |
| | | | 4.12 | 19.23 | 4.12 | 11.05 | | | | |
| | | | -12.10 | 4.12 | -5.19 | 1.71 | | | | |

| **Maximum Design Pile Load (tons/pile) =** | **48.53** |
|---|---|
| **Tension (tons/pile) =** | **-12.10** |
| ***Pile Dimension :*** | 9 14" CIP |
| ***Footing Dimension :*** **13.00' L x 13.00' W x 4.00' D** | |

tension

Pmax (1) = Pmax/# of Pile + My/Sy + Mz/Sz
Pmax (2) = Pmax /# of Pile + Me/min(Sz,Sy)
Pmax (3) = Pmax/# of Pile + Me/Sy + Mz/Sz
Pmax (4) = Pmax/# of Pile + My/Sy + Me/Sz

Pmin (1) = Pmin/# of Pile - My/Sy - Mz/Sz
Pmin (2) = Pmin /# of Pile - Me/min(Sz,Sy)
Pmin (3) = Pmin/# of Pile - Me/Sy - Mz/Sz
Pmin (4) = Pmin/# of Pile - My/Sy - Me/Sz

**PILE FOOTING DESIGN** (AASHTO 4.4) SP File:Pseicft21bt23 Date: 08/24/06 49

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Bridge | EXAMPLE1 (MO) | County | Pemiscot | Column # | 1 | | |
| Bent # | 2 | | | | | | |
| # of columns = | 3 | *Calculate FWS reduction factor:* | | Footing volume (ft.^3) = | 676 | Footing weight (kips) = | 101.40 |
| Column diameter (ft.) = | 3 | roadway width (ft.) = | 38.83333 | Add. column Ht. (ft.)* = | 0 | Fill weight = | 58.30 |
| Fill depth (ft.) = | 3 | 1/2 of adjacent spans (ft.) = | 60 | | | Add. column weight = | 0.00 |
| **Number of piles** =** | 9 (14" CIP) | FWS reduction (k/col.) = | 27.18 | | | Max add. (ftg+col+fill) = | 159.70 |
| Footing depth (ft.) = | 4 | | | | | Min Add. (ftg+col-FWS) = | 74.22 |
| Ft length along Z axis (ft.) = | 13 (*OR*) | Mini. Ftg length (ft.)*** = | | | | Min eccentricity (ft), e' = | 0.14 |
| Ft width along Y axis (ft) = | 13 (*OR*) | Mini. Ftg width (ft.)*** = | | **W/Seismic** Max | **46.90** Ton/pile | | |
| | | | | **W/Seismic** Min | **-10.72** Ton/pile | | |

Group sect. mod., Sz = 30
Group sect. mod., Sy = 30
30
30
Lenght to widht ratio = 1.00

AASHTO LRFD Table 3.4.1-1
Load Combination & Load factors

Note: Sz and Sy are group section modulus based on spacing ratio, and equal to sum of centroid dist. times # of piles divided by centroid dist.
where Sz is about bridge direction.

**Recommended LRFD Guidelines section 6.4.2 & 8.5** for seismic design
Plie size allowed 16" Used 14" CIP
Cant'r dist. From face of column to edge of pile cap, Lftg = 5
Ratio of Lftg/Ftg depth = 1.25 <= 2.5 If Ratio of Lftg/Ftg depth >2.5 then use modification factor = 1.2
Factor 1
Mp(o) = Mp * Factor

Seismic Load comb

| | Long dir. | Trans dir. |
|---|---|---|
| Factor | 0.7 | 0.7 |

**Group 7 = 1.00 (Earth quake)**

where,
e (y) is along footing lenght
e (z) is along footing width

Pmax = Axial + (Max. add) / load factor
Pmin = Axial + (Min. add) / load factor
Me = Pmax * e'

| Mp(o) | Load Group | Load Factor | Axial | My | Mz | Pmax | Me | Pmax (1) | Pmax (2) | Pmax (3) | Pmax (4) | Pmin | M'e | Pmin (1) | Pmin (2) | Pmin (3) | Pmin (4) | TENSION PER PILE (TONS) | Axial | Mp | Mp(o) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Four possible load cases (Tons) | | | | | | Four possible load cases (Tons) | | | | | | | |
| | SEISMIC | | | | | | | | | | | | | | | | | | | | |
| 0 | Column1 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| 910 | Column1 | 1.0 | 177 | 837 | 637 | 336.70 | 0.00 | 39.94 | 18.71 | 29.32 | 29.32 | 251.22 | 0.00 | -7.28 | 13.96 | 3.34 | 3.34 | -7.28 | 177 | 910 | 910 |
| 0 | Column2 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| 829 | Column2 | 1.0 | 81 | 580.3 | 580.3 | 240.70 | 0.00 | 32.72 | 13.37 | 23.04 | 23.04 | 155.22 | 0.00 | -10.72 | 8.62 | -1.05 | -1.05 | -10.72 | 81 | 829 | 829 |
| 0 | Column3 | 1.0 | 0 | 0 | 0 | 159.70 | 0.00 | 8.87 | 8.87 | 8.87 | 8.87 | 74.22 | 0.00 | 4.12 | 4.12 | 4.12 | 4.12 | | 0 | 0 | 0 |
| 982 | Column3 | 1.0 | 272 | 687.4 | 687.4 | 431.70 | 0.00 | 46.90 | 23.98 | 35.44 | 35.44 | 346.22 | 0.00 | -3.68 | 19.23 | 7.78 | 7.78 | -3.68 | 272 | 982 | 982 |
| | **for seismic analysis.** | | | | 46.90 | SEISMIC | MAXIMUM | 46.90 | 23.98 | 35.44 | 35.44 | | | 4.12 | 19.23 | 7.78 | 7.78 | | | | |
| | | | | | -10.72 | SEISMIC | MINIMUM | 8.87 | 8.87 | 8.87 | 8.87 | | | -10.72 | 4.12 | -1.05 | -1.05 | | | | |

**Maximum Design Pile Load (tons/pile) = 46.90**
**Tension (tons/pile) = -10.72** tension
***Pile Dimension :*** **9 14" CIP**
***Footing Dimension :*** **13.00' L x 13.00' W x 4.00' D**

Pmax (1) = Pmax/# of Pile + My/Sy + Mz/Sz
Pmax (2) = Pmax /# of Pile + Me/min(Sz,Sy)
Pmax (3) = Pmax/# of Pile + Me/Sy + Mz/Sz
Pmax (4) = Pmax/# of Pile + My/Sy + Me/Sz

Pmin (1) = Pmin/# of Pile - My/Sy - Mz/Sz
Pmin (2) = Pmin /# of Pile - Me/min(Sz,Sy)
Pmin (3) = Pmin/# of Pile - Me/Sy - Mz/Sz
Pmin (4) = Pmin/# of Pile - My/Sy - Me/Sz

## Pile cap Design

Seismic Load, $P := 114\ \frac{kip}{pile}$ Col dia, $D := 3$ ft

Eq. col, $De := \left(\pi \cdot \frac{D^2}{4}\right)^{0.5}$ $\frac{De}{2} = 1.33$ ft

Footing size:

Width in trans dir., $B := 13$ ft Length in long dir., $L := 13$ ft Footing ht., $H := 48$ in

| No. of pile along y axis | Pile located from face of col | No. of pile along z axis | Pile located from face of col |
|---|---|---|---|
| $N_{L1} := 3$ | $L1 := 3.7$ ft | $N_{B1} := 3$ | $B1 := 3.7$ ft |
| $N_{L2} := 3$ | $L2 := 0$ ft | $N_{B2} := 3$ | $B2 := 0$ ft |

$Fc := 3$ ksi $Fy := 60$ ksi

$M_y := \frac{1}{B}(N_{L1} \cdot L1 + N_{L2} \cdot L2) \cdot P$ $M_y = 97.3\ \frac{k-ft}{ft}$

$M_z := \frac{1}{L}(N_{B1} \cdot B1 + N_{B2} \cdot B2) \cdot P$ $M_z = 97.3\ \frac{k-ft}{ft}$ Use 1ft width $b := 12$ in

$f_r := 0.24 \cdot \left[\left(\frac{Fc}{1}\right)^{0.5}\right]$ $f_r = 0.416$ ksi LRFD 5.4.2.6

$Ig := \frac{1}{12} \cdot b \cdot H^3$ $Ig = 110592$ $in^4$ $Y_t := \frac{H}{2}$

$S_c := \frac{Ig}{Y_t}$ $S_c = 4608$ $in^3$

$Mcr := S_c \cdot \frac{f_r}{12}$ $Mcr = 159.63\ \frac{k-ft}{ft}$ LRFD 5.7.3..3.2

$1.2Mcr = 191.55\ \frac{k-ft}{ft}$

**Long dir bar**

$1.33 \cdot M_y = 129.46\ \frac{k-ft}{ft}$

Use $M_{ydesign} := \min(1.2Mcr, 1.33M_y)$ $M_{ydesign} = 129.46\ k - \frac{ft}{ft}$ Cover $C := 4$ In

Long bar # $Lbar := 7$ Long bar dia $Db_L := \frac{Lbar}{8}$ $As_{Lbar} := \frac{\pi}{4} \cdot {Db_L}^2$ $As_{Lbar} = 0.601\ in^2$

$\phi := 0.9$ $\quad dc_L := H - C - \frac{Db_L}{2}$ $\quad dc_L = 43.56$ in LRFD 5.5.4.2

$Ru_y := M_{ydesign} \cdot \frac{12}{\phi \cdot b \cdot dc_L^{\ 2}}$ $\quad Ru_y = 0.076$

$Rw_y := \frac{0.85 \cdot Fc}{Fy} \cdot \left[1 - \left(1 - 2\frac{Ru_y}{0.85Fc}\right)^{0.5}\right]$ $\quad Rw_y = 0.0013$

As req'd per ft = $As_l := Rw_y \cdot b \cdot dc_L$ $\quad As_l = 0.671 \ \frac{in^2}{ft}$

**2#6** Hair pin bar area, $As_{Eb} := 0.88$

Tot As req'd in long dir, $As_L := As_l \cdot B - As_{Eb}$ $\quad As_L = 7.837 \ in^2$

No. of long bar req'd, $Nb_L := \frac{As_L}{As_{Lbar}}$ $\quad Nb_L = 13$

Use 14-#7 in long dir with **Hair pin bar**

**Trans. dir bar**

$1.33 \cdot M_z = 129.46 \ \frac{k - ft}{ft}$

Use $M_{zdesign} := \min(1.2Mcr, 1.33M_z)$ $\quad M_{zdesign} = 129.46 \frac{k - ft}{ft}$

trans bar # $Tbar := 7$ trans bar dia $Db_T := \frac{Tbar}{8}$ $\quad As_{Tbar} := \frac{\pi}{4} \cdot Db_T^{\ 2}$ $\quad As_{Tbar} = 0.601 \ in^2$

$dc_T := H - C - Db_L - \frac{Db_T}{2}$ $\quad dc_T = 42.69$ in

$Ru_z := M_{zdesign} \cdot \frac{12}{\phi \cdot b \cdot dc_T^{\ 2}}$ $\quad Ru_z = 0.079$

$Rw_z := \frac{0.85 \cdot Fc}{Fy} \cdot \left[1 - \left(1 - 2\frac{Ru_z}{0.85Fc}\right)^{0.5}\right]$ $\quad Rw_z = 0.0013$

As req'd per ft = $As_t := Rw_z \cdot b \cdot dc_T$ $\quad As_t = 0.685 \ \frac{in^2}{ft}$

Tot As req'd in long dir, $As_T := As_t \cdot L - As_{Eb}$ $\quad As_T = 8.021 \ in^2$

No. of long bar req'd, $Nb_T := \dfrac{As_T}{As_{Tbar}}$ $Nb_T = 13$

Use 14-#7 in Trans dir with **Hair pin bar**

Long dir

$N_L := 14$ $As_{Eb} = 0.88$

$$As_{Ltot} := \frac{N_L \cdot As_{Lbar} + As_{Eb}}{B}$$

$As_{Ltot} = 0.715 \ \frac{in^2}{ft}$

$$a_L := As_{Ltot} \cdot \frac{Fy}{0.85 \cdot Fc \cdot b}$$

$a_L = 1.402$ in

$dc_L = 43.563$ in

$$Mr_y := \phi \cdot As_{Ltot} \cdot \frac{Fy}{12 \cdot 1} \cdot \left(dc_L - \frac{a_L}{2}\right)$$

$Mr_y = 138 \ \frac{k-ft}{ft}$

$$\text{LongReinforcement} := \begin{cases} \text{" O. K. "} & \text{if } M_{ydesign} < Mr_y \\ \text{" N. G. "} & \text{otherwise} \end{cases}$$

LongReinforcement = " O. K. "

Trans dir

$N_T := 14$ $As_{Eb} = 0.88$

$$As_{Ttot} := \frac{N_T \cdot As_{Tbar} + As_{Eb}}{L}$$

$As_{Ttot} = 0.715 \ \frac{in^2}{ft}$

$$a_T := As_{Ttot} \cdot \frac{Fy}{0.85 \cdot Fc \cdot b}$$

$a_T = 1.402$ in

$dc_T = 42.688$ in

$$Mr_z := \phi \cdot As_{Ttot} \cdot \frac{Fy}{12 \cdot 1} \cdot \left(dc_T - \frac{a_T}{2}\right)$$

$Mr_z = 135 \ \frac{k-ft}{ft}$

$$\text{TransReinforcement} := \begin{cases} \text{" O. K. "} & \text{if } M_{ydesign} < Mr_z \\ \text{" N. G. "} & \text{otherwise} \end{cases}$$

TransReinforcement = " O. K. "

**Check One-way Action** LRFD 5.8.3.3

Critical shear location @ dv

$dc := \min(dc_L, dc_T)$ $a := \max(a_L, a_T)$

$dc - \frac{a}{2} = 41.986$ $0.9 \cdot dc = 38.419$ $0.72 \cdot H = 34.56$

$$dv := \max\left[\left(dc - \frac{a}{2}\right), (0.9 \cdot dc), (0.72 \cdot H)\right]$$ $dv = 41.986$ in

$$Vr1 := \phi \cdot 0.25 \cdot \frac{Fc}{1} \cdot (B \cdot 12) \cdot dv$$ $Vr1 = 4421$ kip $\beta := 2$

$Vr2 := \phi \cdot 0.0316 \cdot (B \cdot 12) \cdot dv \cdot \beta \cdot \left(\frac{Fc}{1}\right)^{0.5}$ $Vr2 = 645$ kip

$Vr := \min(Vr1, Vr2)$

$Vr = 645$ kip

Critical shear location $\frac{De}{2} + \frac{dv}{12} = 4.83$ ft from cent of column

No. of pile out side critical shear location, $N_{pile} := 3$

Shear load $V := N_{pile} \cdot P$ $V = 342$ kip

$$Oneway shear := \begin{cases} "O.K." & \text{if } V < Vr \\ "N.G." & \text{otherwise} \end{cases}$$

Onewayshear = " O. K. "

**Check Two-way action**

LRFD 5.13.3.6.3

Critical section width, $Cs := \frac{dv}{12} + De$

$bo := 4 \cdot Cs \cdot 12$ $bo = 296$ inch

$\beta c := \frac{De}{De}$

$$Vn := \min\left[\left[\left(0.063 + \frac{0.126}{\beta c}\right) \cdot \left(\frac{Fc}{1}\right)^{0.5} \cdot bo \cdot dv\right], \left[0.126 \cdot \left(\frac{Fc}{1}\right)^{0.5} \cdot bo \cdot dv\right]\right]$$

$Vn = 2708$ kip

$VR := \phi \cdot Vn$ $VR = 2437$ kip

No of pile outside critical area, $N_{pileout} := 8$

$V_{punch} := N_{pileout} \cdot P$ $V_{punch} = 912$ kip

$$Twoway shear := \begin{cases} "O.K." & \text{if } V_{punch} < VR \\ "N.G." & \text{otherwise} \end{cases}$$

Twowayshear = " O. K. "

Shrinkage and temperature reinf. LRFD 5.10.8.2

$$Ag := H \cdot b$$

$$0.11 \cdot \frac{Ag}{Fy} = 1.056 \qquad 0.0015 \cdot Ag = 0.864$$

$$As_{shr} := \min\left[\left(\frac{0.11 \cdot Ag}{Fy}\right), 0.0015 \cdot Ag\right]$$

$$As_{shr} = 0.864$$

$$As_{top} := \frac{As_{shr}}{2}$$

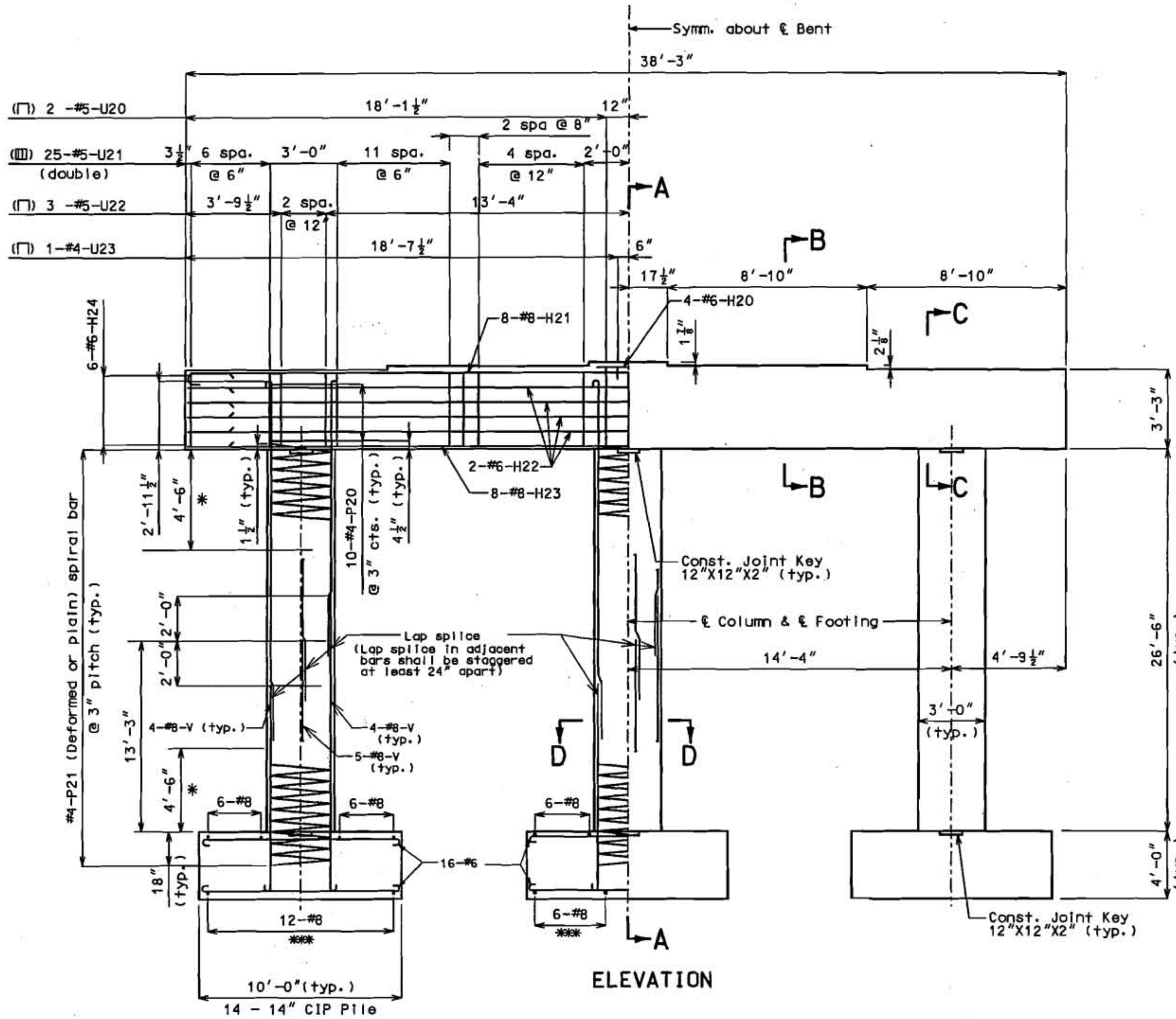
Required As for shrinkage @ top & bottom in either dir $As_{top} = 0.432 \ \frac{in^2}{ft}$

As required by design in either dir, $As := \min(As_l, As_t)$ $As = 0.671 \ \frac{in^2}{ft}$

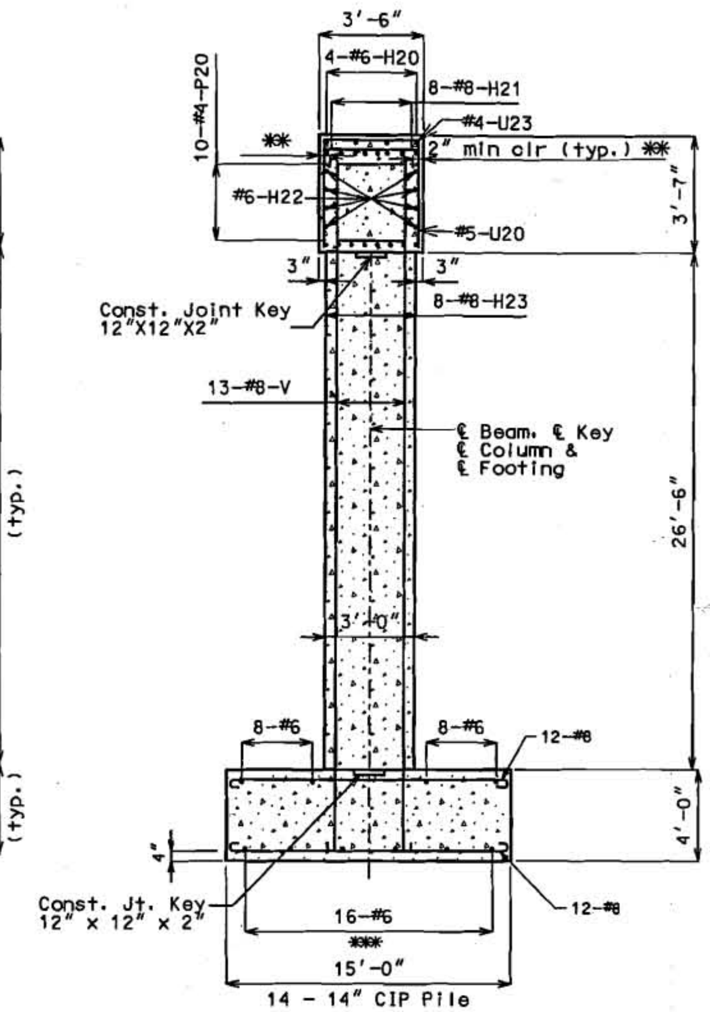
$$ShrinkageReinf := \begin{vmatrix} \text{" O. K. "} & \text{if } As_{top} < As \\ \text{" N. G. "} & \text{otherwise} \end{vmatrix}$$

$$ShrinkageReinf = \text{" O. K. "}$$

# Appendix




ELEVATION

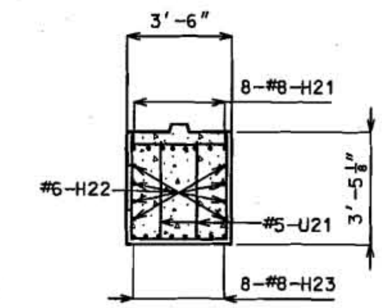



SECTION A-A

NOTE: Spiral P-bar not shown for clarity.

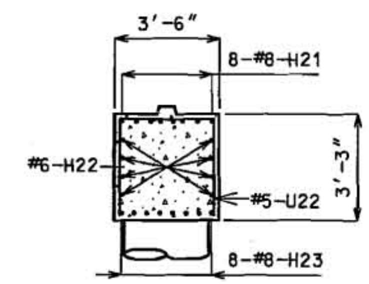
Lap splice of V-bar not shown for clarity.

** If 2" min. clearance do not meet then at the contractor's option, the hooks of V-Bars embedded in beam cap may be oriented inward.

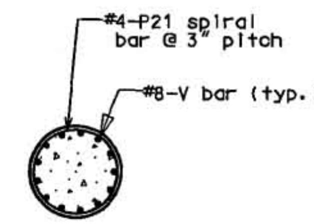



SECTION B-B

NOTE: Dowel bar not shown for clarity.




SECTION C-C




SECTION D-D

NOTE:

* Lapping of spiral reinf. (P-bar) not allowed in this area.

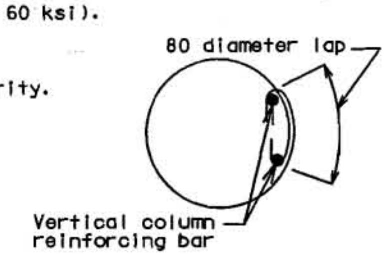
*** Spaced as shown in the footing plan.

Reinforcing steel shall be Grade 60 (fy = 60 ksi).

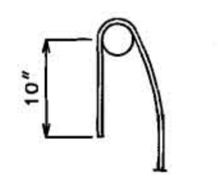
14"Ø CIP piles not shown for clarity.

#6-U-bar in the footing not shown for clarity.

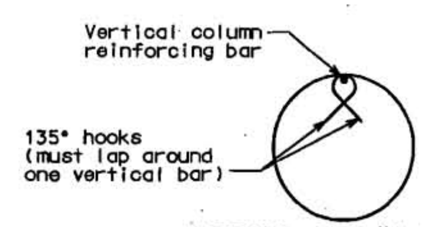
For plan of footing, see next sheet.




ANCHOR SPLICES IN SPIRAL AROUND VERTICAL BAR
(USE FOR INTERMEDIATE SPLICES OF SPIRALS)




DETAILS OF 135° SEISMIC SPIRAL TIE HOOK



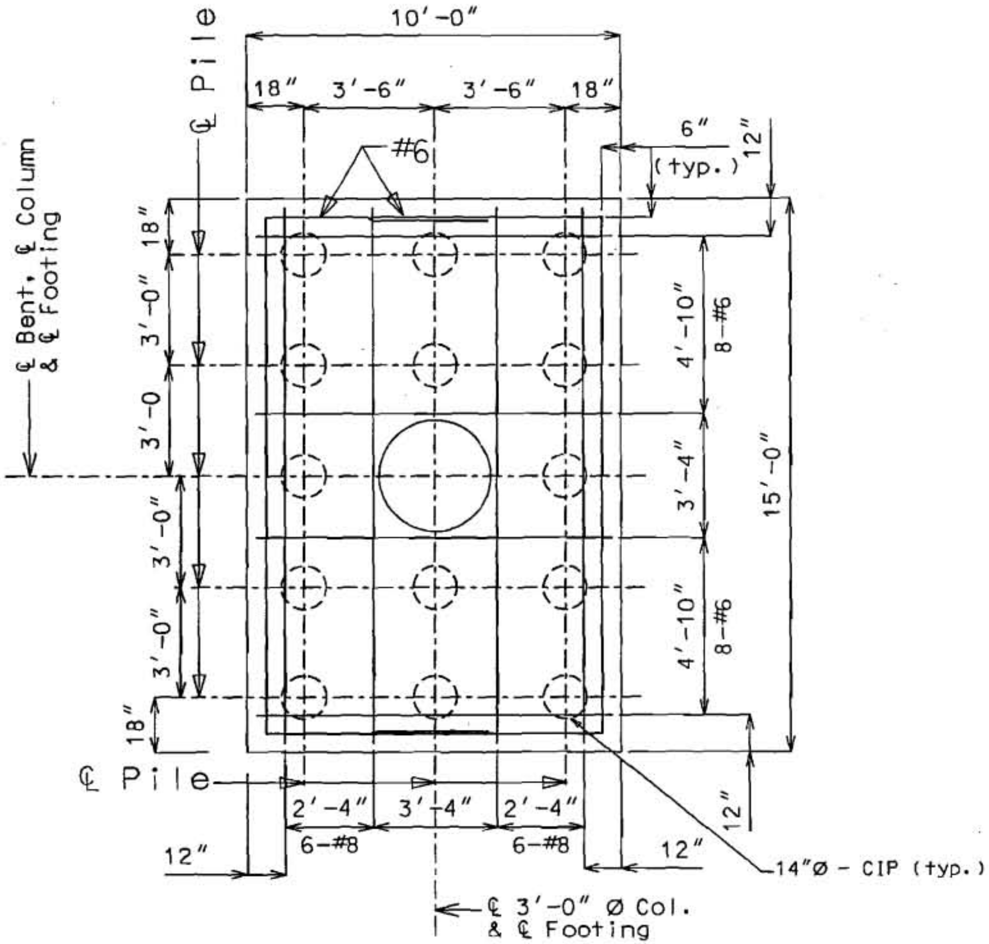

DETAIL OF #4-P20 SEISMIC STIRRUP BAR

Missouri Example 1

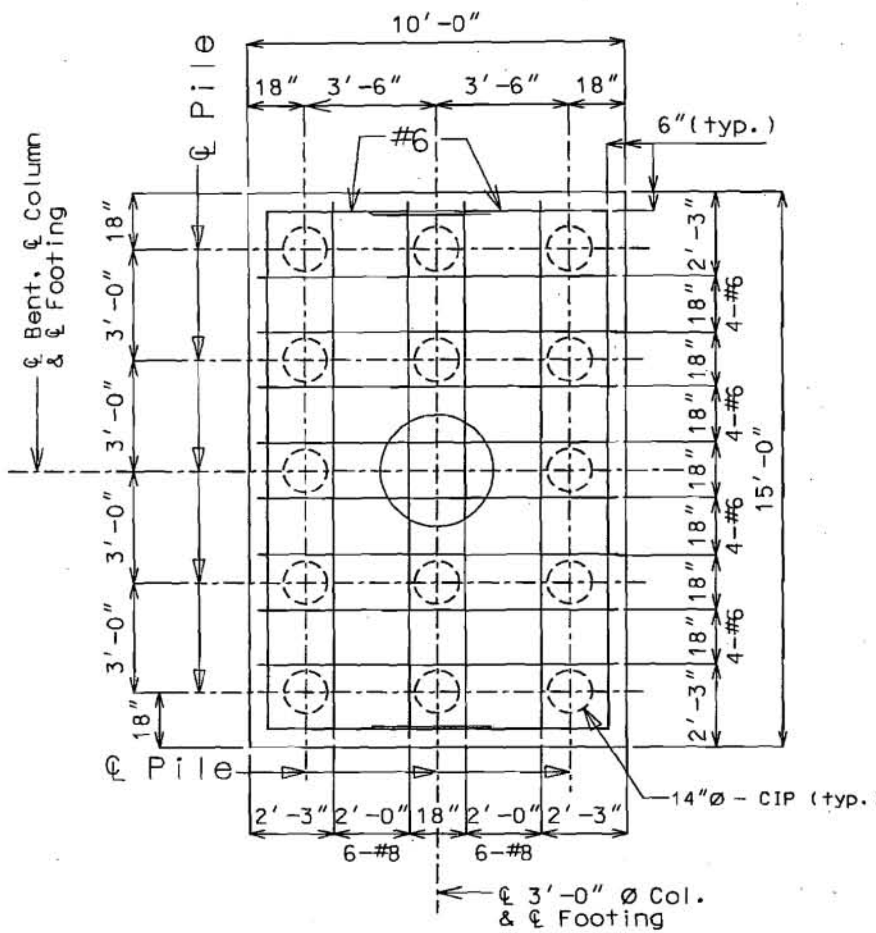
DETAILS OF INTERMEDIATE BENT NO. 2 & 3

500 - Yr EQ

1




PLAN OF FOOTING
SHOWING TOP REINF.



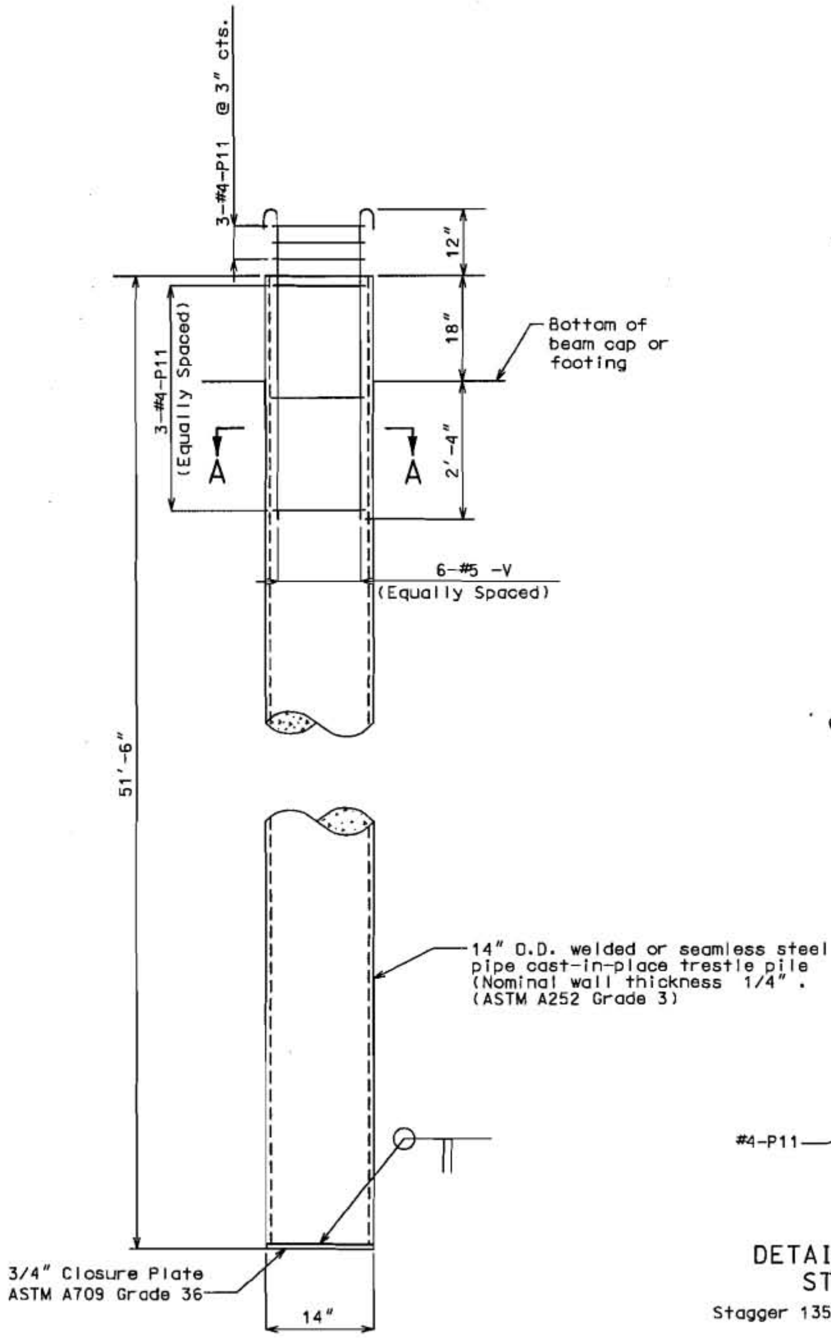

PLAN OF FOOTING
SHOWING BOTTOM REINF.

## DETAILS OF INTERMEDIATE BENT NO. 2 & 3

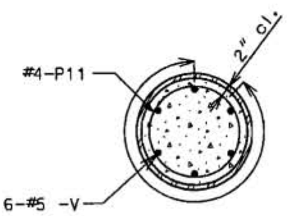
Note: For bent elevation, see previous sheet.

Missouri Example 1

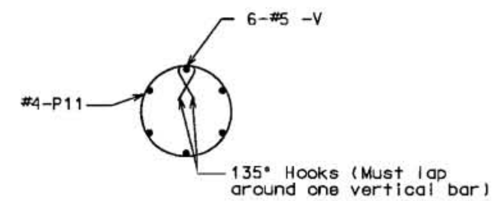
500 - Yr EQ




WELDED OR SEAMLESS STEEL PIPE
CAST-IN-PLACE PILE



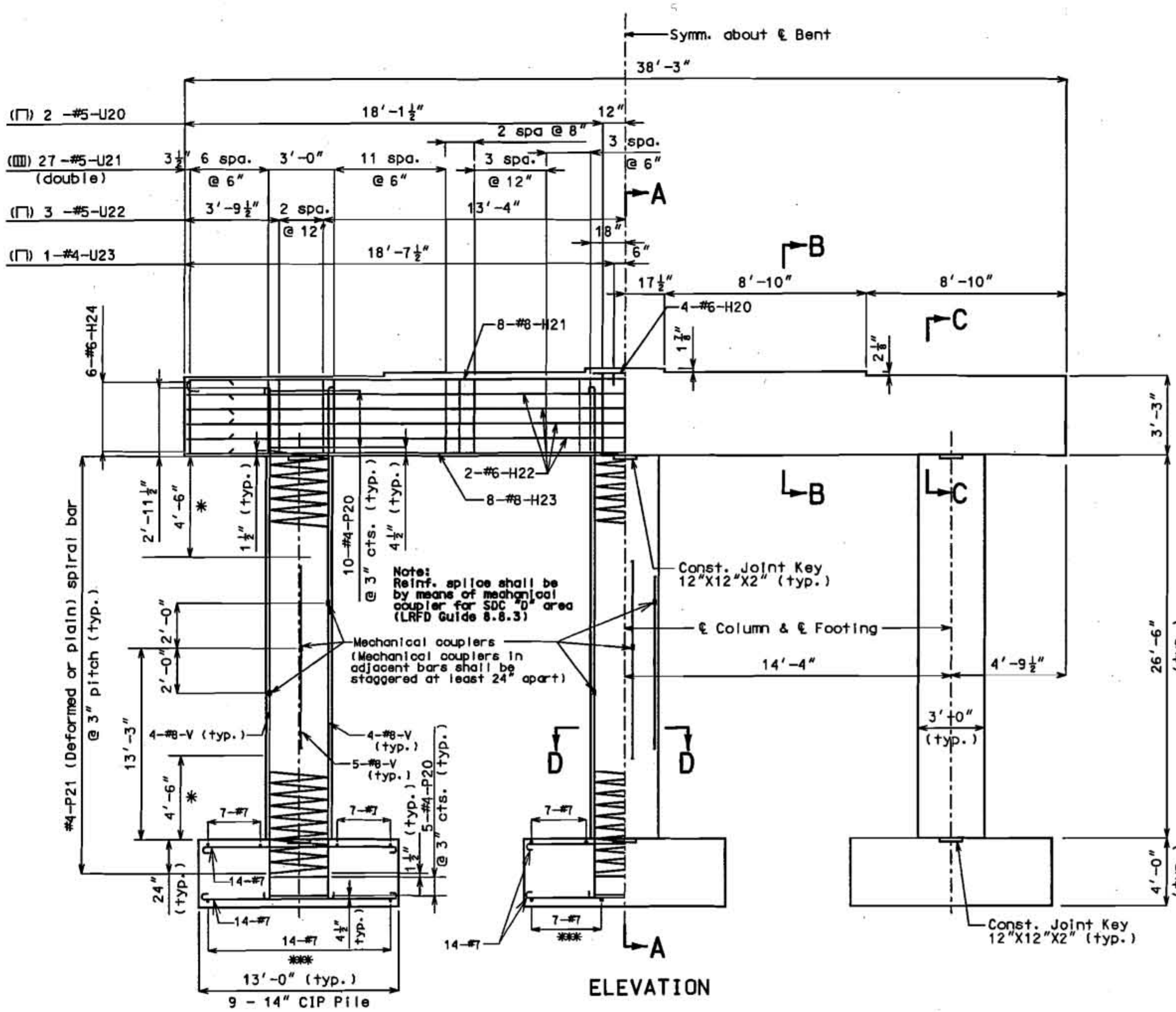

SECTION A-A



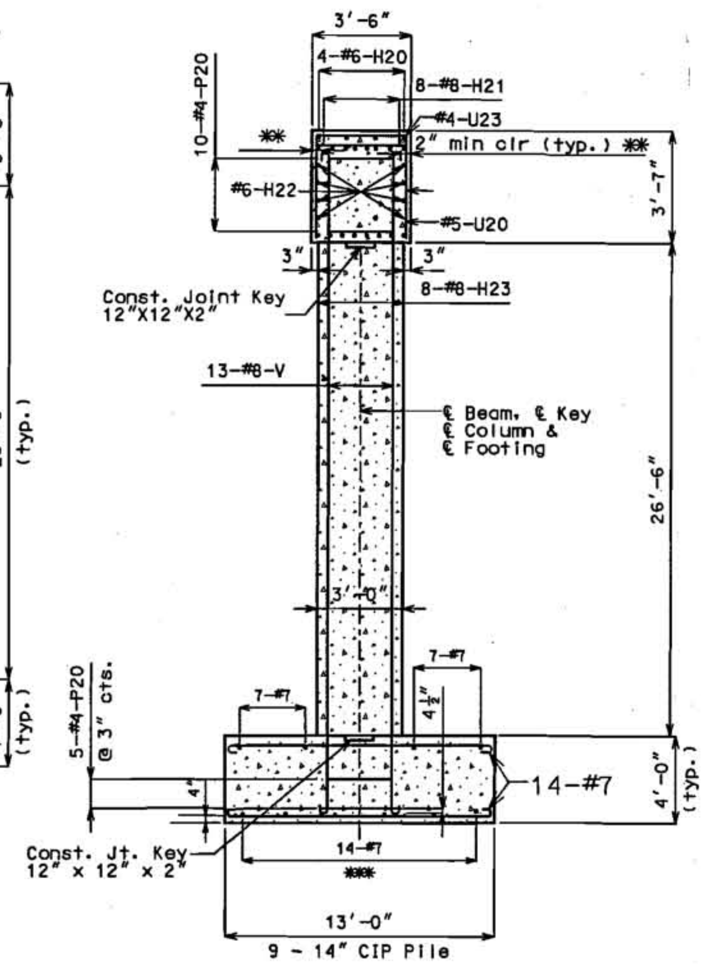

DETAIL OF SEISMIC
STIRRUP BAR

Stagger 135° hook locations for #4-P bar.

Missouri Example 1 DETAILS OF CAST-IN-PLACE PILE 500 - YR EQ

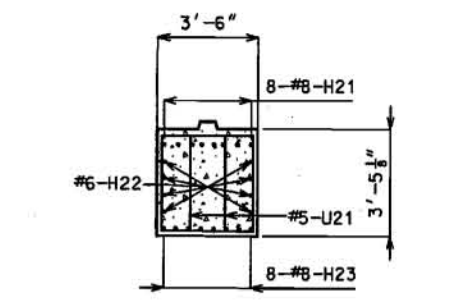



ELEVATION



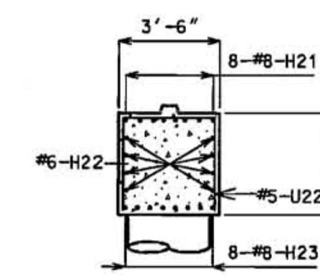

SECTION A-A

NOTE: Spiral P-bar not shown for clarity.
Mechanical couplers not shown for clarity.
*** If 2" min. clearance do not meet then at the contractor's option, the hooks of V-Bars embedded in beam cap may be oriented inward.

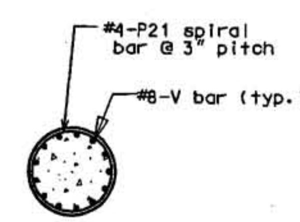



SECTION B-B

NOTE: Dowel bar not shown for clarity.




SECTION C-C




SECTION D-D

NOTE :

* Lapping of spiral reinf. (P-bar) not allowed in this area.

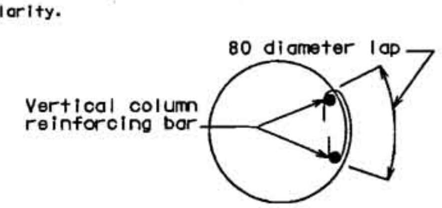
Reinforcing steel shall be A706 (fy = 60 ksi).

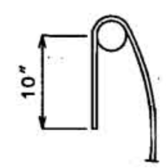
14"Ø CIP piles not shown for clarity.

#6 -U-bar in the footing not shown for clarity.

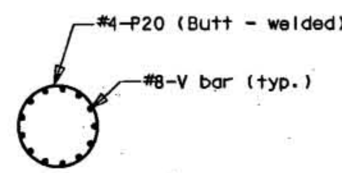
For plan of footing, see next sheet.

*** Spaced as shown in the footing plan.




DETAILS OF 135° SEISMIC SPIRAL TIE HOOK




ANCHOR SPLICES IN SPIRAL AROUND VERTICAL BAR
(USE FOR INTERMEDIATE SPLICES OF SPIRALS)

DETAIL OF #4-P20
SEISMIC STIRRUP (HOOP) BAR

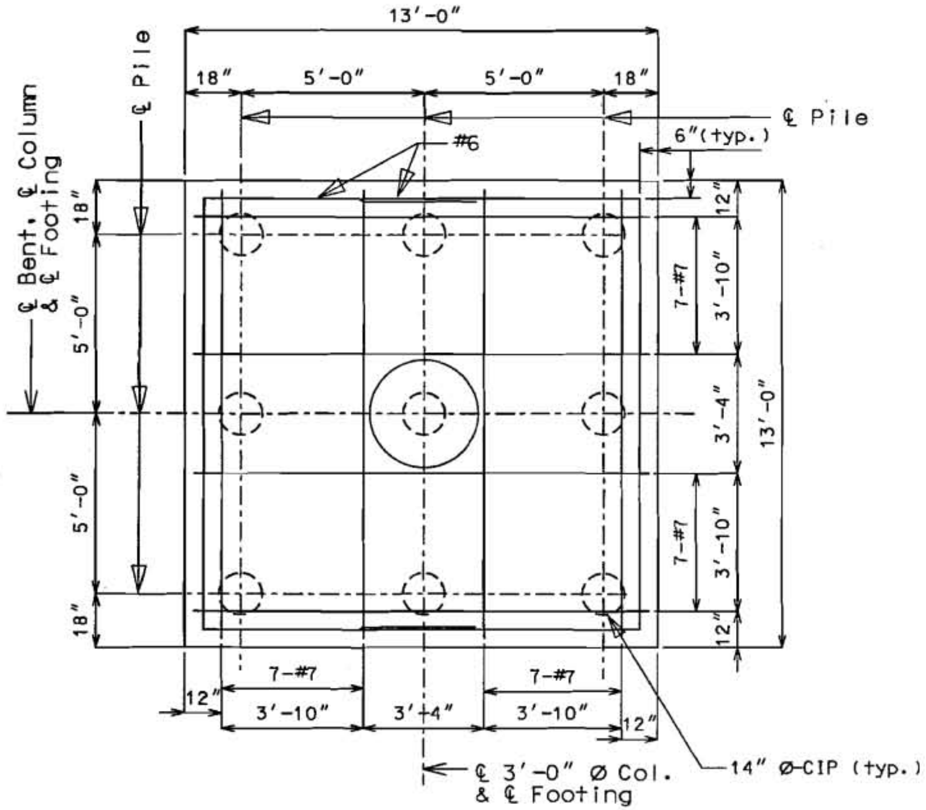
NOTE :
500 - Yr to 1000 - Yr detail changes shown with the "BLUE" color.

Missouri Example 1

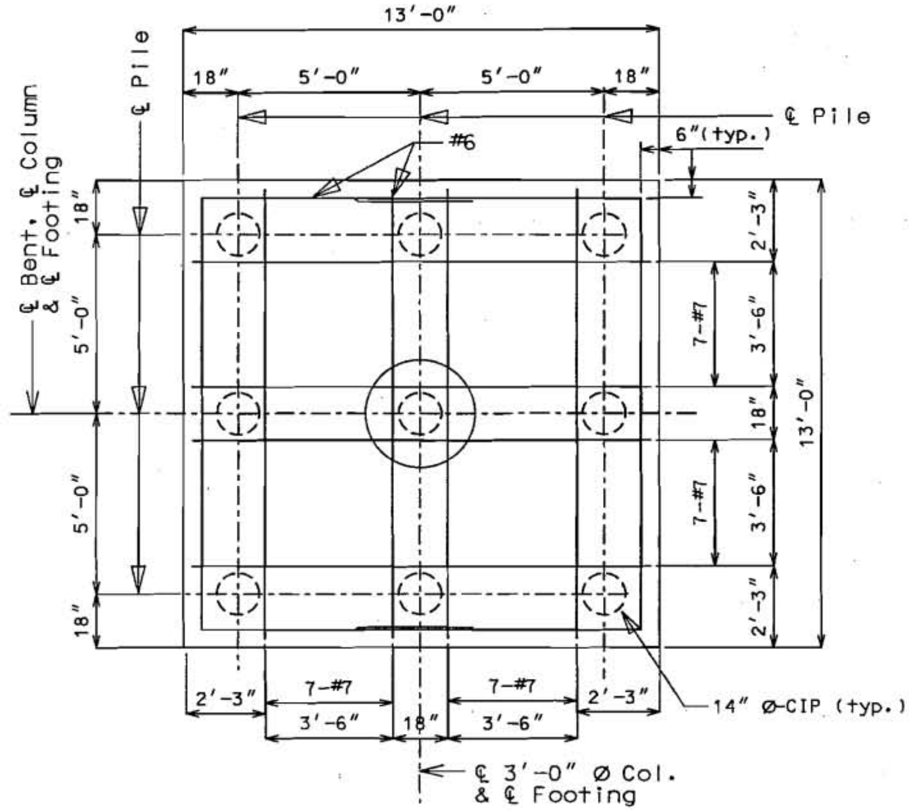
DETAILS OF INTERMEDIATE BENT NO. 2 & 3

1000 - YR EQ

4




PLAN OF FOOTING
SHOWING TOP REINF.



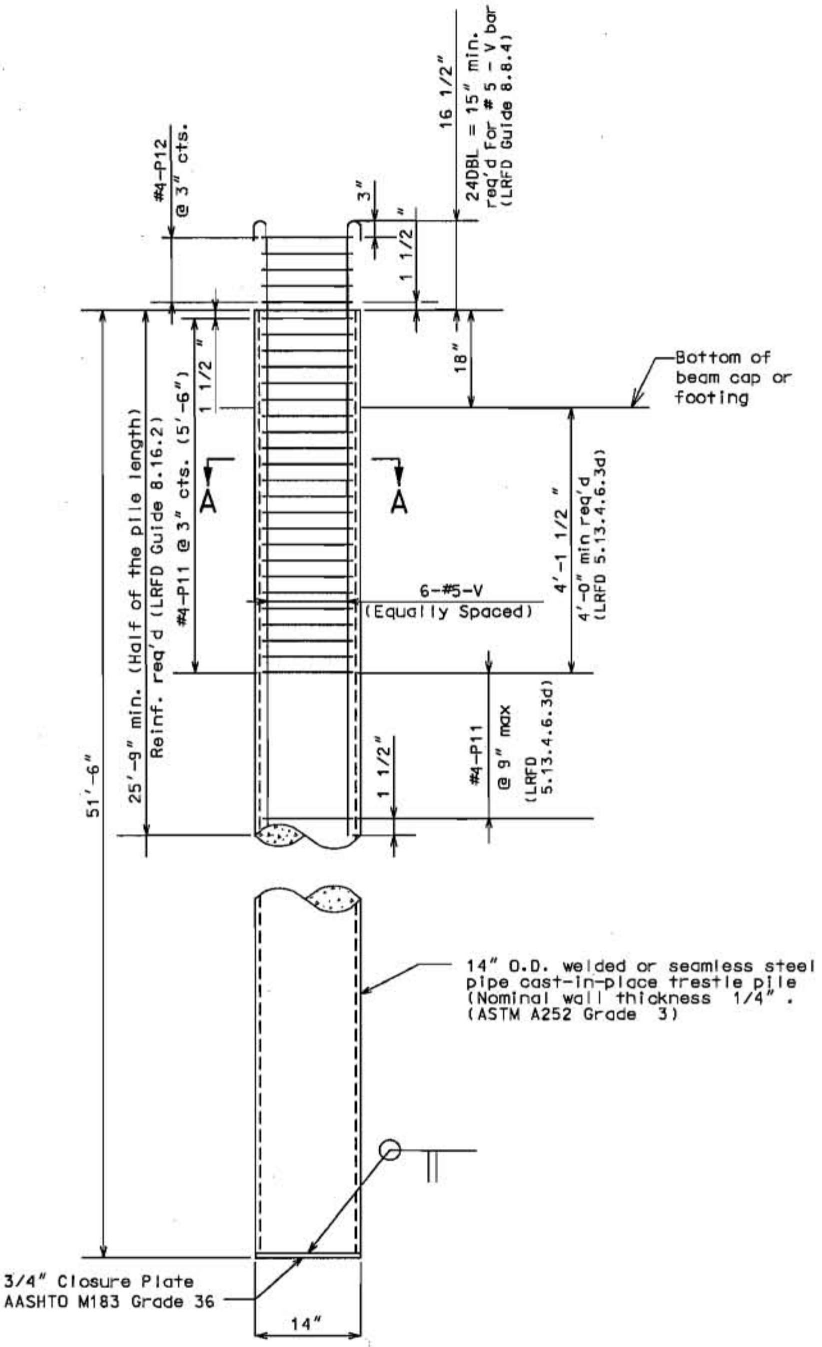

PLAN OF FOOTING
SHOWING BOTTOM REINF.

# DETAILS OF INTERMEDIATE BENT NO. 2 & 3

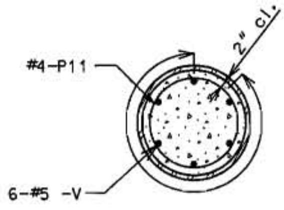
Note: For bent elevation, see previous sheet.

Missouri Example 1

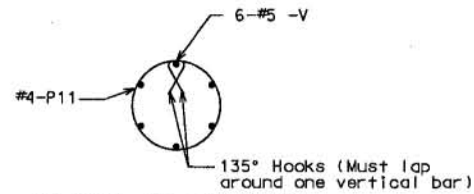
1000 - Yr EQ



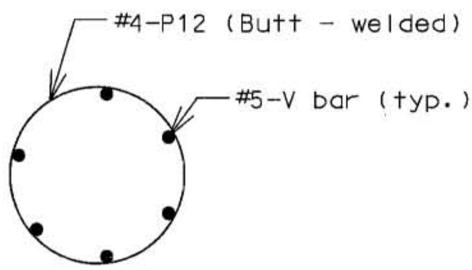

WELDED OR SEAMLESS STEEL PIPE
CAST-IN-PLACE PILE




SECTION A-A




DETAIL OF SEISMIC
STIRRUP BAR
Stagger 135° hook locations for #4-P bar.




DETAIL OF #4-P12
SEISMIC STIRRUP (HOOP) BAR

NOTE :
500 - Yr to 1000 - Yr
detail changes
shown with the
"BLUE" color.

Missouri Example 1 DETAILS OF CAST-IN-PLACE PILE 1000 - YR EQ