TRIAL DESIGN BRIDGE DESCRIPTION

State: *Tennessee*

Trial Design Designation: *TN-3*

Bridge Name: *Holmes Street overCSX Railroad*

Superstructure Type: *Prestressed precast AASHTO Type II I-beams, composite concrete deck*

Span Length(s): *Three spans @40ft.-60ft.-48ft.*

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

Foundation: *Prestressed concrete friction piles*

Abutments: *Integral on prestressed concrete friction piles*

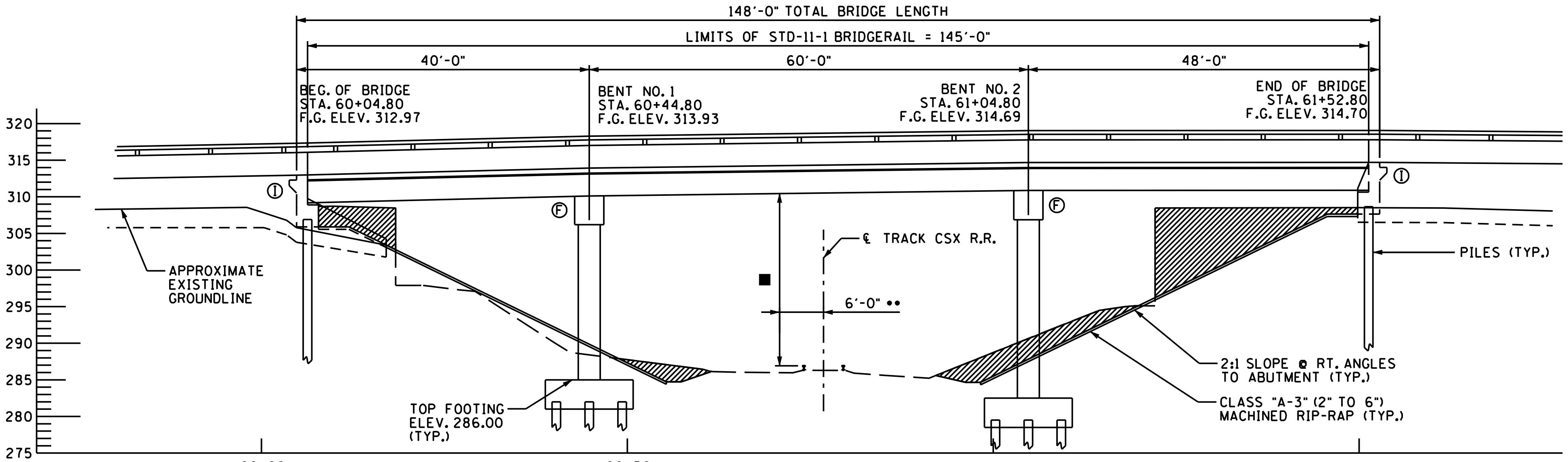
Seismic Design Category (SDC): "C"

Additional Description (Optional): ______________________________

PE. NO.	79960-1526-94	
PROJECT NO.	YEAR	SHEET NO.
BRZE-9409 (85)	2007	

REVISIONS

NO.	DATE	BY	BRIEF DESCRIPTION



•• DENOTES MEASURED PERPENDICULAR TO ℄ TRACK

■ ACT. VERT. CL. = 23'-2⅝" MIN REQD VERT. CL. = 23'-0"

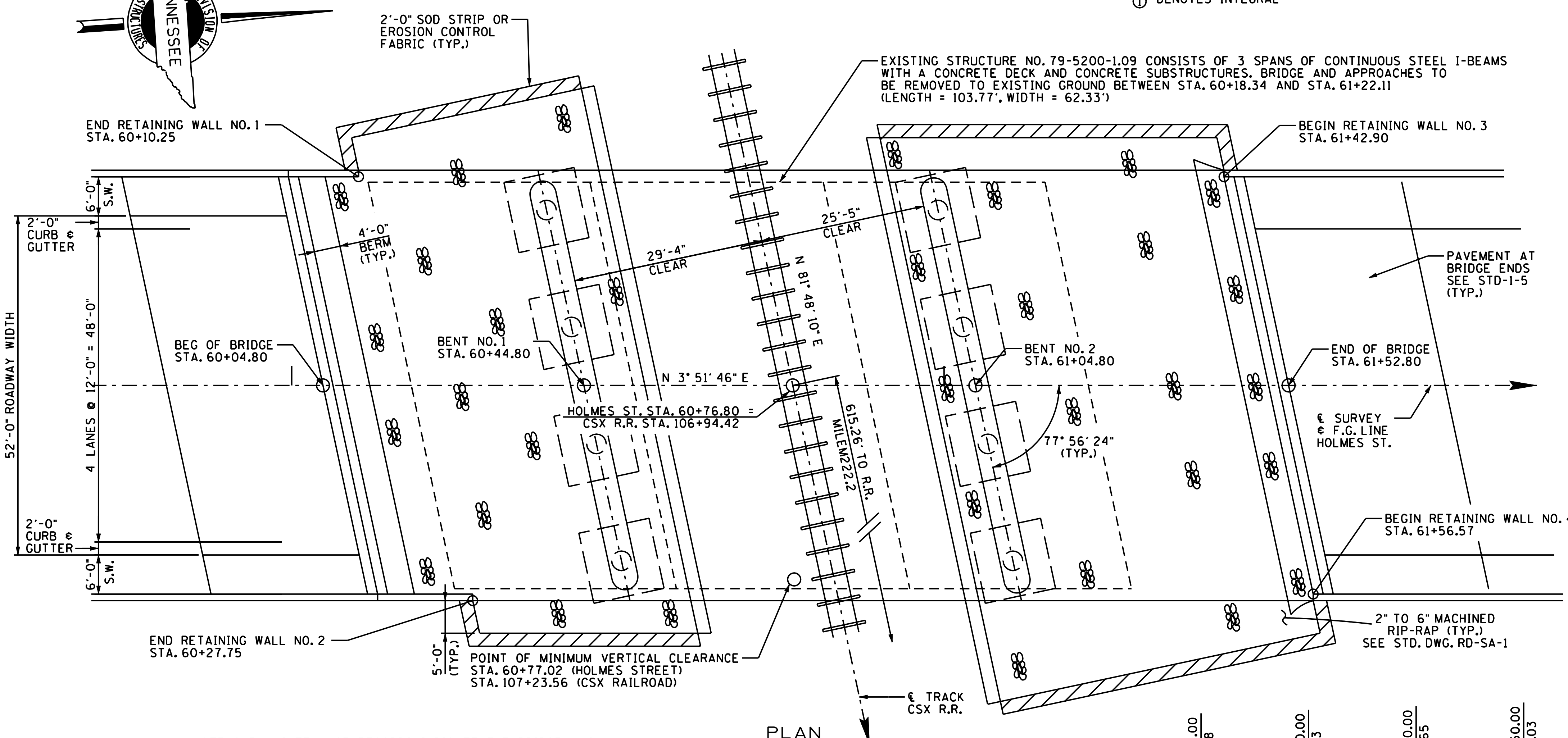
ELEVATION

SCALE 1" = 10'-0"

DENOTES AREA TO BE EXCAVATED AND PAID FOR AS A ROADWAY ITEM

(F) DENOTES FIXED

(I) DENOTES INTEGRAL



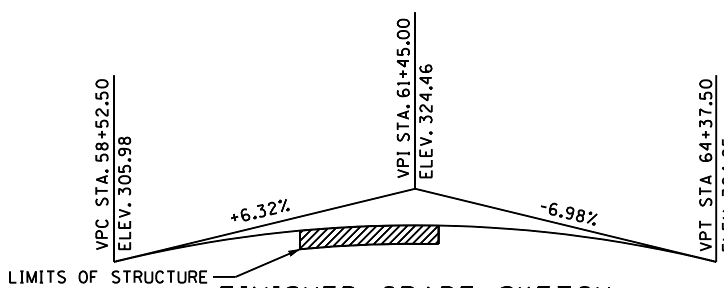
PLAN

SCALE 1" = 10'-0"

NOTE: OUR MAINTENANCE RECORDS INDICATE THE BRIDGE WAS ORIGINALLY PAINTED WITH MATERIALS CONTAINING LEAD AND/OR CHROMATES AND THE CONTRACTOR IS REQUIRED TO PROCEED ACCORDINGLY TO TAKE ALL MANDATORY SAFEGUARDS PRESCRIBED BY STATE AND FEDERAL LAW FOR BOTH WORKER PROTECTION AND HAZARDOUS MATERIALS DISPOSAL.

GENERAL NOTES

1) SPECIFICATIONS: STANDARD ROAD AND BRIDGE SPECIFICATIONS OF THE TENNESSEE DEPARTMENT OF TRANSPORTATION (MARCH 1, 2006 EDITION.)
2) DESIGN SPECIFICATIONS: AASHTO LRFD 2004 EDITION WITH ADDENDA INCLUDING THE STANDARD SPECIFICATIONS FOR SEISMIC DESIGN OF HIGHWAY BRIDGES. (SEISMIC PERFORMANCE ZONE 3 WITH ACCELERATION COEFFICIENT 0.20).
3) LOADING: HL-93 (DEAD LOADS TO INCLUDE 35 PSF FOR FUTURE OVERLAY).
4) CONCRETE: CLASS "A" F'c = 3000 PSI; BRIDGE DECK, CLASS "D" F'c = 4000 PSI.
5) CLASS "D" CONCRETE FOR BRIDGE DECKS SHALL BE IN ACCORDANCE WITH SECTION 604 OF THE STANDARD SPECIFICATIONS.
6) REINFORCING STEEL: TO BE ASTM A615 GRADE 60. (EPOXY COAT ALL SLAB STEEL).
7) SUPERSTRUCTURE: TO CONSIST OF 3 SPAN CONTINUOUS AASHTO TYPE II I-BEAM WITH COMPOSITE CONCRETE DECK SLAB.
8) USE STD-11-1 PARAPET.
9) RIP-RAP: MACHINED RIP-RAP SHALL BE CLASS "A-3" AND SHALL BE 2" TO 6" IN SIZE UNIFORMLY GRADED AND MEET THE QUALITY REQUIREMENT OF SUBSECTION 918-10 AND SHALL BE MEASURED AND PAID FOR UNDER ROADWAY ITEMS.
10) TEXTURE COATING: TO BE MOUNTAIN GREY (36440); EXCEPT TRAFFIC FACE AND TOP OF PARAPET TO BE PAINTED WHITE (37886).
11) BRIDGE EXCAVATION: BASED ON FINAL PROFILE AT ABUTMENTS AND BENTS.
12) BRIDGE DECK FINISH TO BE IN ACCORDANCE WITH NOTE "C" IN ARTICLE 2604.22 OF THE STANDARD SPECIFICATIONS.
13) BRIDGE DECK DRAINS ARE NOT REQUIRED.
14) END OF BRIDGE DRAINS ARE NOT REQUIRED.
15) CLOSE ROAD DURING CONSTRUCTION.
16) EXISTING STRUCTURE NO. 79-5200-1.09 TO BE REMOVED TO EXISTING GROUND BETWEEN STA. 60+18.34 AND STA. 61+22.11.



FINISHED GRADE SKETCH

(HOLMES ST.)

2026 ADT = 11,830
52'-0" ROADWAY W/ 6'-0" SIDEWALKS & STD-11-1 PARAPET
DESIGN SPEED = 40 MPH

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

PRELIMINARY LAYOUT
HOLMES ST. OVER CSX RAILROAD
BRIDGE ID NO. 79OB6370001
STATION 60+76.80 L.M. 1.09
SHELBY COUNTY
2007

GRADE SKETCH

(CSX R.R.)

CORRECT Edward P. Wasserman
ENGINEER OF STRUCTURES

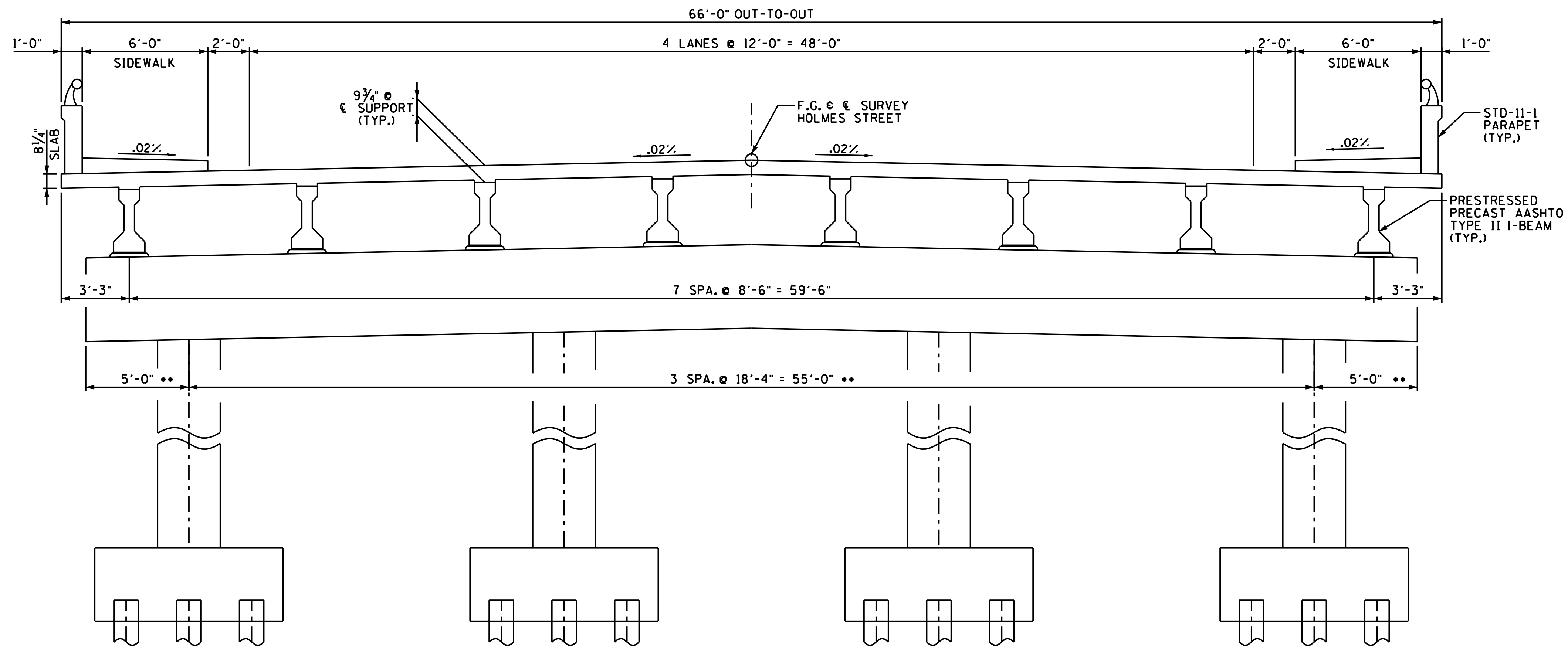
DESIGNED BY	P. CHAMBERS	DATE	5-05
DRAWN BY	P. CHAMBERS	DATE	5-05
SUPERVISED BY	KDM	DATE	5-05
CHECKED BY		DATE	

CLASS "A-3" RIP-RAP = 330 TONS

PE. NO.	79960-1526-94	
PROJECT NO.	YEAR	SHEET NO.
BRZE-9409 (85)	2007	

REVISIONS

NO.	DATE	BY	BRIEF DESCRIPTION



TYPICAL CROSS SECTION
(LOOKING FORWARD ON SURVEY)

•• DENOTES MEASURED ALONG ℄ OF BENT

2026 ADT = 11,830
52'-0" ROADWAY W/ 6'-0" SIDEWALKS & STD-11-1 PARAPET
DESIGN SPEED = 40 MPH

STATE OF TENNESSEE
DEPARTMENT OF TRANSPORTATION

PRELIMINARY LAYOUT
HOLMES ST. OVER CSX RAILROAD
BRIDGE ID NO. 79OB6370001
STATION 60+76.80 L.M. 1.09
SHELBY COUNTY
2007

CORRECT Edward P. Wasserman
ENGINEER OF STRUCTURES

DESIGNED BY P. CHAMBERS DATE 5-05
DRAWN BY P. CHAMBERS DATE 5-05
SUPERVISED BY KDM DATE 5-05
CHECKED BY DATE

3.2 PERFORMANCE CRITERIA

Bridges shall be designed for the life safety performance objective considering a one level design for a 5% probability of exceedance in 50 years. Higher levels of performance, such as the operational objective, may be used with the authorization of the bridge owner. Development of design earthquake ground motions for the 5% probability of exceedance in 50 years are given in Article 3.4.

This analysis will be for 5% probability of exceedence in 50 years, Previous design was according to current specifications based on design requirements (LRFD 2004).

3.3 EARTHQUAKE RESISTING SYSTEMS (ERS) REQUIREMENTS FOR SDC C & D

For SDC C or D (see Article 3.5), all bridges and their foundations shall have a clearly identifiable Earthquake Resisting System (ERS) selected to achieve the Life Safety Criteria defined in Section 3.2. The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the surrounding soil and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen design strategy of seismic resistance and other structural requirements.

This is what TDOT considers its ERS:

Transverse or
Longitudinal Response

- Abutment required to resist the design earthquake elastically
- Longitudinal passive soil pressure must be less than 0.70 of the value obtained using the procedure given in Article 5.2.3

3.4 SEISMIC GROUND SHAKING HAZARD

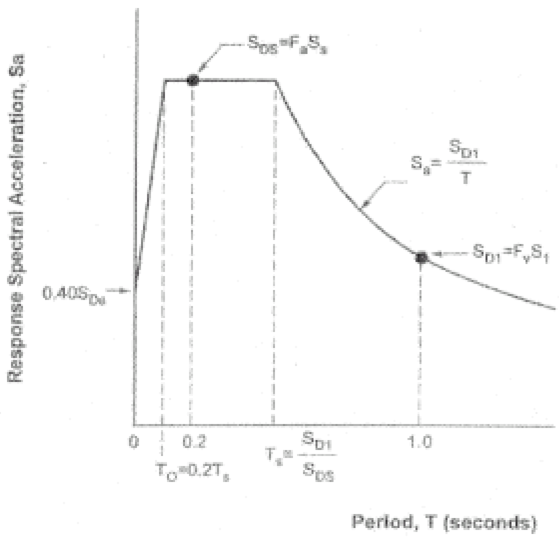
The ground shaking hazard prescribed in these Specifications is defined in terms of acceleration response spectra and site coefficients. They shall be determined in accordance with the general procedure of Section 3.4.1 or the site-specific procedure of Section 3.4.3.

In the general procedure, the spectral response parameters are defined using the USGS/AASHTO seismic hazard maps produced by the U.S. Geological Survey depicting probabilistic ground motion and spectral response for 5% probability of exceedance in 50 years.

The response spectra and site coefficients will be determined in accordance with the general procedure, rather than site-specific procedure.

3.4.1 Design Spectra Based on General Procedure

Design response spectra shall be constructed using response spectral accelerations taken from national ground motion maps described in this section and site factors described in Article 3.4.2. The construction of the response spectra shall follow the procedures described below and illustrated in Figure 3.4.1-1.



Design earthquake response spectral acceleration at short periods, S_{DS}, and at 1 second period, S_{DI}, shall be determined from Equations 3.1 and 3.2, respectively:

$$S_{DS} = F_a S_s \tag{3.1}$$

and

$$S_{DI} = F_v S_I \tag{3.2}$$

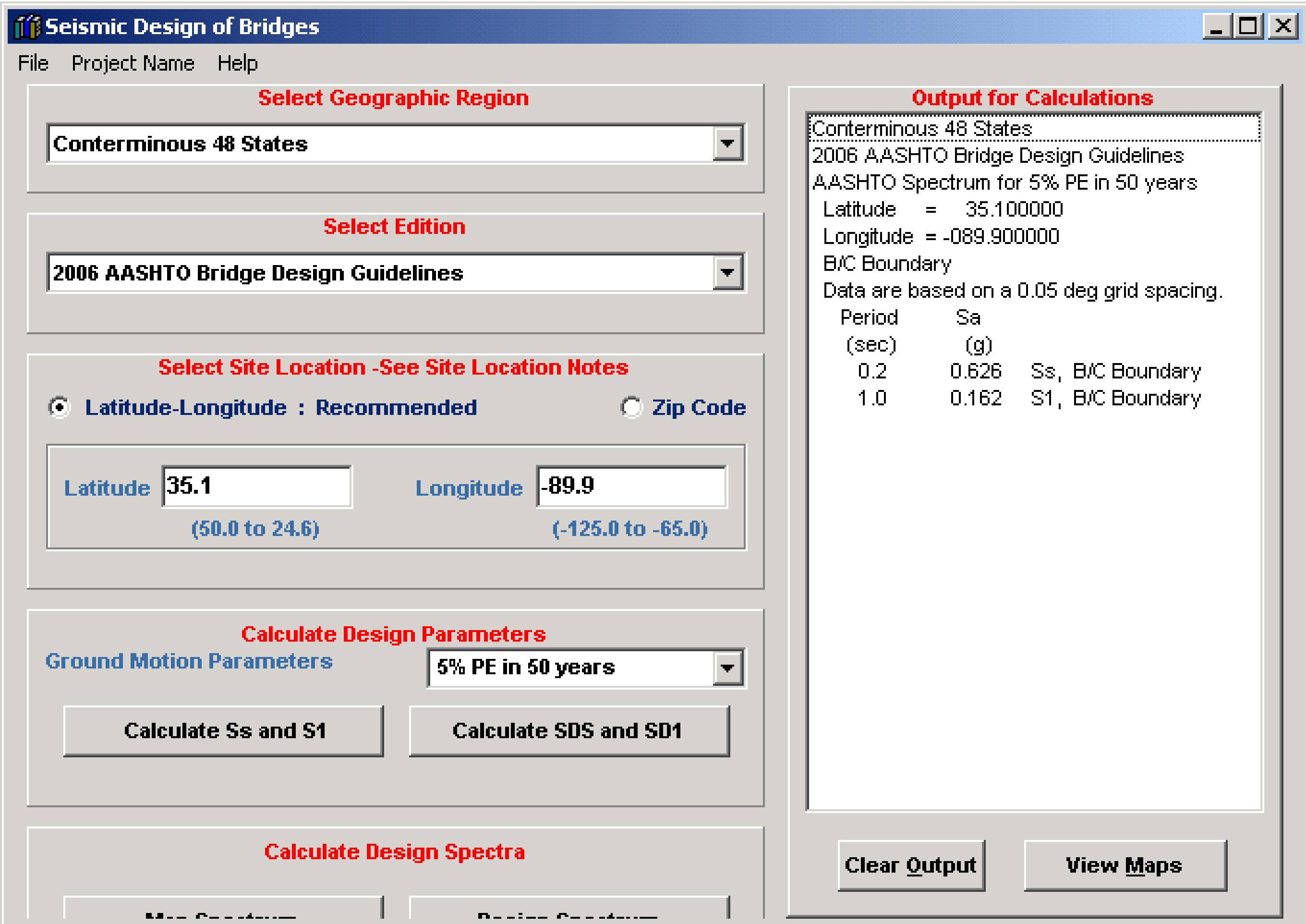
where S_s and S_1 are the 0.2-second period spectral acceleration and 1-second period spectral acceleration, respectively, on Class B rock from ground motion maps described below and F_a and F_v are site coefficients described in Article 3.4.2.3. Values of S_s and S_1 may be obtained by the following methods:

(a) S_s and S_I may be obtained from national ground motion maps (Figures 3.4.1-2 through 3.4.1-14 at the end of this section).

(b) S_s and S_I may be obtained from the *Seismic Design Parameters* CD-ROM published by the U.S. Geological Survey (USGS) for site coordinates specified by latitude and longitude or zip code.

GoogleEarth was used to obtain longitude and latitude for the side:

These coordinates were input into the Seismic Hazard Curves program to obtain values for S_s and S_1:



These are the values of F_a:

Table 3.4.2.3-1: Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at Short Periods				
	$S_s \le 0.25$ g	$S_s = 0.50$ g	$S_s = 0.75$ g	$S_s = 1.00$ g	$S_s \ge 1.25$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*a*	*a*	*a*	*a*	*a*

Table notes: Use straight line interpolation for intermediate values of S_s, where S_s is the spectral acceleration at 0.2 second obtained from the ground motion maps.

By interpolation, F_a shall be taken to be 1.3 for Site Class D and $S_s = 0.626$

These are the values of F_v:

Table 3.4.2.3-2: Values of F_v as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \le 0.1$ g	$S_1 = 0.2$ g	$S_1 = 0.3$ g	$S_1 = 0.4$ g	$S_1 \ge 0.5$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*a*	*a*	*a*	*a*	*a*

Table notes: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration at 1.0 second obtained from the ground motion maps.

By interpolation, F_v shall be taken to be 2.15 for Site Class D and S_s = 0.162

Thus:

$S_{DS} := 1.3 \cdot 0.626$ $\quad S_{DS} = 0.814$ $\quad S_{D1} := 2.15 \cdot 0.162$ $\quad S_{D1} = 0.348$

1. For periods less than or equal to T_0, the design response spectral acceleration, S_a, shall be defined by Equation 3.3:

$$S_a = 0.60 \frac{S_{DS}}{T_0} T + 0.40 S_{DS} \qquad (3.3)$$

T and T_0 are defined below.

Note that for $T = 0$ seconds, the resulting value of S_a is equal to peak ground acceleration, PGA.

2. For periods greater than or equal to T_0 and less than or equal to T_s, the design response spectral acceleration, S_a, shall be defined by Equation 3.4:

$$S_a = S_{DS} \qquad (3.4)$$

where $T_0 = 0.2 T_s$, and $T_s = S_{D1} / S_{DS}$, and T = period of vibration (sec).

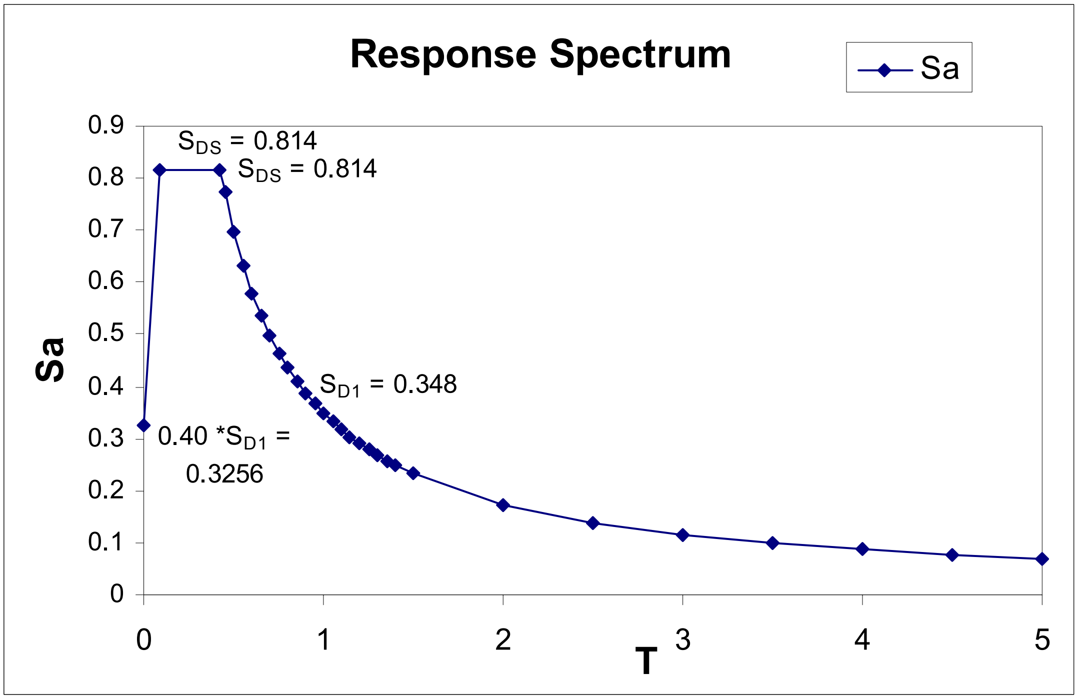
$T_s = \frac{S_{D1}}{S_{DS}}$ $\quad T_s := \frac{0.348}{0.814}$ $\quad T_s = 0.428$

$T_0 := 0.2 \cdot T_s$ $\quad T_0 = 0.086$

3. For periods greater than T_s, the design response spectral acceleration, S_a, shall be defined by Equation 3.5:

$$S_a = \frac{S_{D1}}{T} \tag{3.5}$$

Using the criteria above, this is the response spectrum for this structure:



Compare with the response spectrum from the ATC 6 curve of the current design:

```
   .
0.8+
   !
0.7+
   !
0.6+
   !
0.5+XXXXX
   !     X
0.4+      X
   !       XX
0.3+         XX
   !           XXXX
0.2+               XXXXXXX
   !                      XXXXXXXXXXXXX
0.1+                                   XXXXXXXXXXXXXXXX
   !
0.0+
    +---------+---------+---------+---------+---------+
    0         1         2         3         4         5
                   PERIOD  (SECONDS)
```

3.5 SELECTION OF SEISMIC DESIGN CATEGORY SDC

Each bridge shall be designed to one of four Seismic Design Categories (SDC), A through D, based on the one-second period design spectral acceleration for the Life Safety Design Earthquake (S_{D1} refer to Section 3.4.1) as shown in Table 3.5.1.

Table 3.5.1: Partitions for Seismic Design Categories A, B, C and D

Value of S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \le S_{D1} < 0.30g$	B
$0.30g \le S_{D1} < 0.50g$	C
$0.50g \le S_{D1}$	D

The Seismic Design Category for this structure will be **C**

The five requirements for each of the proposed Seismic Design Categories are shown in Figure 3.5.1 and described below. For both single span bridges and bridges classified as SDC A the connections must be designed for specified forces in Article 4.5 and Article 4.6 respectively, and must also meet minimum support length requirements of Article 4.12.

3. SDC C
   a. Identification of ERS
   b. Demand Analysis
   c. Implicit Capacity Check Required (displacement, $P-\Delta$, seat width)
   d. Capacity Design Required including column shear requirement
   e. SDC C Level of Detailing

The previous design was based on these criteria:

3.10.3 Importance Categories

For the purpose of Article 3.10, the Owner or those having jurisdiction shall classify the bridge into one of three importance categories as follows:

- Critical bridges,
- Essential bridges, or
- Other bridges.

The basis of classification shall include social/survival and security/defense requirements. In classifying a bridge, consideration should be given to possible future changes in conditions and requirements.

This bridge was considered to have an Importance Category of an "other" bridge

3.10.4 Seismic Performance Zones

Each bridge shall be assigned to one of the four seismic zones in accordance with Table 1.

This bridge was previously designed as in accordance with Seismic Performance Zone 3.

Table 3.10.4-1 Seismic Zones.

Acceleration Coefficient	Seismic Zone
A 0.09	1
$0.09 < A$ 0.19	2
$0.19 < A$ 0.29	3
$0.29 < A$	4

3.10.9.4 Seismic Zones 3 and 4

3.10.9.4.1 General

Structures in Seismic Zones 3 and 4 shall be analyzed according to the minimum requirements specified in Articles 4.7.4.1 and 4.7.4.3.

The design forces of each component shall be taken as the lesser of those determined using:

- the provisions of Article 3.10.9.4.2; or
- the provisions of Article 3.10.9.4.3,

for all components of a column, column bent and its foundation and connections.

3.10.9.4.2 Modified Design Forces

Modified design forces shall be determined as specified in Article 3.10.9.3, except that for foundations the R-factor shall be taken as 1.0.

Table 3.10.7.1-1 Response Modification Factors—Substructures.

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers—larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical pile only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

The designer assumed a Response Modification Factor of 5 for the bents for a structure classified as "Other".

Table 3.10.7.1-2 Response Modification Factors—Connections.

Connection	All Importance Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

3.10.5 Site Effects

3.10.5.1 General

Site effects shall be included in the determination of seismic loads for bridges.

The site coefficient, S, specified in Table 1, shall be based upon soil profile types defined in Articles 3.10.5.2 through 3.10.5.5.

This structure was assumed to be in a Soil Profile Type II.

3.10.5.3 Soil Profile Type II

A profile with stiff cohesive or deep cohesionless soils where the soil depth exceeds 200 ft. and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays shall be taken as Type II.

Table 3.10.5.1-1 Site Coefficients.

Site Coefficient	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

Comparing the displacement design criteria between the previous design and the recommended seismic specifications:

Recommended Specifications:

4.3 DETERMINATION OF SEISMIC LATERAL DISPLACEMENTS DEMANDS

The global structure displacement demand, Δ_D^T, is the total seismic displacement at a particular location within the structure or subsystem. The global displacement demand will include components attributed to foundation flexibility, Δ_f (i.e. foundation rotation or translation), flexibility of essentially elastic components such as bent caps Δ_b, and the flexibility attributed to elastic and inelastic response of ductile members Δ_y and Δ_{pd}, respectively.

Minimum requirements for superstructure, abutment, and foundation modeling are specified in Section 5.

4.3.1 Horizontal Ground Motions

For bridges classified as SDC B, C or D the global seismic displacement demands, Δ_D^T, shall be determined independently along two perpendicular axes by the use of the analysis procedure specified in Section 4.2. The resulting displacements shall then be combined as specified in Section 4.4. Typically, the perpendicular axes are the longitudinal and transverse axes of the bridge. The longitudinal axis of a curved bridge may be selected along a chord connecting the two abutments.

4.3.3 Displacement Magnification For Short Period Structures

Displacements calculated from elastic analysis shall be multiplied by the factor R_d obtained from Equation 4.5 to obtain the design displacement demand specified in Article 4.3. This magnification applies in cases where the fundamental period of the structure T is less than the characteristic ground motion period T^*, corresponding to the peak energy input spectrum.

Values T^* are given in Table 4.3.

$$R_d = \left(1 - \frac{1}{R}\right)\frac{T^*}{T} + \frac{1}{R} \geq 1 \quad \text{For } \frac{T^*}{T} \geq 1 \qquad (4.5a)$$

$$R_d = 1 \quad \text{For } \frac{T^*}{T} \leq 1 \qquad (4.5b)$$

Table 4.3 Values of Characteristic Ground Motion Period, T*

Values of T* (in seconds)												
	M_W=6.5±0.25				M_W=7.25±0.25				M_W=8.0±0.25			
$0.4S_S$ (g)	Class B	Class C	Class D	Class E	Class B	Class C	Class D	Class E	Class B	Class C	Class D	Class E
0.1	0.32	0.45	0.46	0.44	0.41	0.53	0.56	0.56	0.51	0.69	0.71	0.71
0.2	0.37	0.44	0.49	0.64	0.42	0.53	0.55	0.74	0.47	0.61	0.65	0.85
0.3	0.35	0.43	0.50	0.73	0.38	0.51	0.55	0.76	0.48	0.64	0.65	0.98
0.4	0.39	0.47	0.50	0.87	0.42	0.56	0.59	0.93	0.46	0.62	0.66	1.04
0.5	0.37	0.46	0.50	-	0.42	0.53	0.62	-	0.45	0.59	0.70	-
0.6	0.35	0.44	0.50	-	0.43	0.54	0.64	-	0.46	0.60	0.76	-
0.7	-	-	-	-	0.50	0.66	0.76	-	0.54	0.71	0.80	-

For the analysis, a value of M_w equal to 7.25 is assumed.

To obtain the fundamental period in each direction:

```
                    VIBRATION CHARACTERISTICS

                  PARTICIPATION FACTORS       % OF TOTAL MASS
MODE PERIOD  CS     Long    Vert    Tran    Long    Vert    Tran
---- ------ ---- ------- ------- ------- ------- ------- -------
   1  0.362 0.81   1.788  -0.022   8.476   4.157   0.001  93.426
   2  0.266 0.81  -0.010  -0.004   0.699   4.157   0.001  94.061
   3  0.250 0.81   8.440  -0.119  -1.768  96.803   0.019  98.127
   4  0.180 0.81   0.158   2.520  -0.021  96.836   8.278  98.128
   5  0.129 0.81   0.039  -5.353  -0.020  96.838  45.540  98.128
   6  0.101 0.81   0.135   4.709  -0.015  96.861  74.377  98.128
   7  0.070 0.72  -0.043  -0.546   0.004  96.864  74.764  98.128
   8  0.049 0.60  -0.009  -1.004   0.007  96.864  76.076  98.129
   9  0.041 0.56   0.006   0.220   0.103  96.864  76.139  98.142
```

For the longitudinal direction, the fundamental period is taken to be 0.25 sec
For the transverse direction, the fundamental period is taken to be 0.362 sec

$S_S := 0.626$ $\quad 0.4 \cdot S_S = 0.25$ $\quad$ interpolating from above chart: $T' := 0.52\text{sec}$

In the longitudinal direction, T*/T = 2.08 > 1
In the transverse direction, T*/T = 1.43 > 1

Thus: $$R_d = \left(1 - \frac{1}{R}\right)\cdot\frac{T'}{T} + \frac{1}{R}$$ For SDC C, the value of R is taken to be 3

For the longitudinal direction:

$$R_{dL} := \left(1 - \frac{1}{3}\right)\cdot 2.08 + \frac{1}{3} \qquad R_{dL} = 1.72$$

For the transverse direction:

$$R_{dT} := \left(1 - \frac{1}{3}\right)\cdot 1.43 + \frac{1}{3} \qquad R_{dT} = 1.287$$

These are the unmagnified displacements:

BENT CQC DISPLACEMENTS

		....LEFT FACE....		....RGHT FACE....		...OPNNG/CLSNG...	
ITEM	LC	LNGTUDNL	TRNSVRSE	LNGTUDNL	TRNSVRSE	LNGTUDNL	TRNSVRSE
BNT 2	1	0.041	0.018	0.041	0.018	0.000	0.000
	2	0.019	0.079	0.019	0.079	0.000	0.000
	3	0.046	0.041	0.046	0.041	0.000	0.000
	4	0.031	0.084	0.031	0.084	0.000	0.000
BNT 3	1	0.041	0.020	0.041	0.020	0.000	0.000
	2	0.019	0.088	0.019	0.088	0.000	0.000
	3	0.047	0.046	0.047	0.046	0.000	0.000
	4	0.031	0.094	0.031	0.094	0.000	0.000

$$\Delta_L := 0.047\text{ft} \qquad \Delta_T := 0.094\text{ft}$$

$$\Delta_{LD} := \Delta_L \cdot R_{dL} \qquad \Delta_{LD} = 0.081\,\text{ft} \qquad \Delta_{LD} = 0.97\,\text{in}$$

$$\Delta_{LT} := \Delta_T \cdot R_{dT} \qquad \Delta_{LT} = 0.121\,\text{ft} \qquad \Delta_{LT} = 1.451\,\text{in}$$

4.8 STRUCTURE DISPLACEMENT CAPACITY FOR SDC B, C, AND D

For SDC B, C and D, each bridge bent or frame shall satisfy Equation 4.6.

$$\Delta^L_D < \Delta^L_C \quad (4.6)$$

where

Δ^L_D is the displacement demand along the local principal axes of a ductile member resulting from a seismic motion applied to the total structural system according to Article 4.4.

Δ^L_C is the corresponding member displacement capacity available along the same axis as the displacement demand Δ^L_D.

The formulas presented below are used to obtain Δ^L_C for SDC B and C. A more detailed push-over analysis is required to obtain Δ^L_D for SDC D as described in Article 4.8.2 below.

4.8.1 Local Displacement Capacity for SDC B and C

For SDC B and C, the displacement capacity, Δ^L_C, of each bent shall be implicitly calculated respectively based on:

For SDC B

$$\Delta^L_C(ft) = \frac{H_o}{100} * (-1.27 * Ln(x) - 0.32) \geq \frac{H_o}{100} \quad (4.7a)$$

For SDC C

$$\Delta^L_C(ft) = \frac{H_o}{100} * (-2.32 * Ln(x) - 1.22) \geq \frac{H_o}{100} \quad (4.7b)$$

where

$$x = \Lambda \, B_o / H_o \quad (4.7c)$$

Λ is a fixity factor for the column equal to:

$\Lambda = 1$ for fixed-free (pinned on one end).

a. $\Lambda = 2$ for fixed top and bottom.

For a partially fixed connection on one end, interpolation between 1 and 2 is permitted.

B_o = Column Width or Diameter (ft.).

H_o = Height from top of footing to top of the column (i.e., column clear height, ft.).

$\Lambda := 2$

$H_o := 21.59ft$

$B_o := 3ft$

$x := \Lambda \cdot \frac{B_o}{H_o}$ $\quad x = 0.278 \quad \ln(x) = -1.28$

$\Delta_{LC} := \frac{H_o}{100} \cdot (-2.32 \cdot \ln(x) - 1.22)$

$\frac{H_o}{100} = 0.216\,ft \qquad \frac{H_o}{100} = 2.591\,in$

$\Delta_{LC} = 0.378\,ft \qquad \Delta_{LC} = 4.536\,in$

$\Delta_{LT} = 1.451\,in \qquad \frac{\Delta_{LC}}{\Delta_{LT}} = 3.125$

Structure is adequate for implicit displacement capacity demands for SCD C.

4.11.5 P-Δ Capacity Requirement for SDC C & D

The dynamic effects of gravity loads acting through lateral displacements shall be included in the design. The magnitude of displacements associated with P-Δ effects can only be accurately captured with non-linear time history analysis. In lieu of such analysis, P-Δ effects can be ignored if Equation 4.9 is satisfied:

$$P_{dl} \times \Delta_r \le 0.25 \times M_p \text{ for concrete members} \quad (4.9a)$$

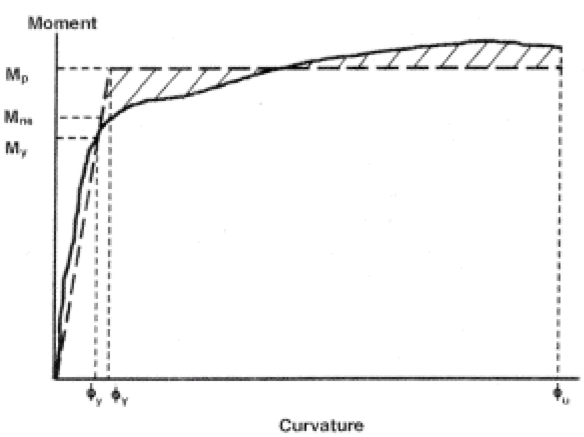
$$P_{dl} \times \Delta_r \le 0.25 \times M_n \text{ for steel members} \quad (4.9b)$$

where:

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

8.5 PLASTIC MOMENT CAPACITY FOR DUCTILE CONCRETE MEMBERS SDC B, C, AND D

The plastic moment capacity of all ductile concrete members shall be calculated by moment-curvature $(M-\phi)$ analysis based on the expected material properties. Moment curvature analysis derives the curvatures associated with a range of moments for a cross section based on the principles of strain compatibility and equilibrium of forces. The $M-\phi$ curve can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve should pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity is obtained by equating the areas between the actual and the idealized $M-\phi$ curves beyond the first reinforcing bar yield point. See Figure 8.4.



$kip := 1000lbf$

From RC Pier Output:

$P_c := 67.94kip + 176.6kip \qquad P_c = 244.54\,kip$

$P_w := 30.46kip$

$P_{dl} := 0.9 \cdot P_c + 0.65 \cdot P_w \qquad P_{dl} = 239.885\,kip$

BENT CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	....LEFT FACE.... TRNSVRSE	....RGHT FACE.... LNGTUDNL	....RGHT FACE.... TRNSVRSE
BNT 2	1	0.041	0.018	0.041	0.018
	2	0.019	0.079	0.019	0.079
	3	0.046	0.041	0.046	0.041
	4	0.031	0.084	0.031	0.084
BNT 3	1	0.041	0.020	0.041	0.020
	2	0.019	0.088	0.019	0.088
	3	0.047	0.046	0.047	0.046
	4	0.031	0.094	0.031	0.094

$\Delta_{rL} := 0.047ft \qquad \Delta_{rT} := \frac{0.094ft}{2}$

$\Delta_{rL} = 0.564\,in \qquad \Delta_{rT} = 0.564\,in$

8.4.2 Reinforcing Steel Modeling

Reinforcing steel shall be modeled with a stress-strain relationship (see Figure 8.1) that exhibits an initial elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain. Within the elastic region the modules of elasticity, E_s, shall be 29,000 ksi. For SDC D A706 reinforcing steel shall be used with the following expected properties:

$$f_{ye} = 1.1 f_y \qquad (8.1)$$

where

f_{ye} = the expected yield strength

f_y = the specified minimum yield strength

$$f_{ue} = 1.4\ f_{ye} \qquad (8.2)$$

where

f_{ue} is the expected tensile strength

The ultimate tensile strain ε_{su} shall be:

$$\varepsilon_{su} = \begin{cases} 0.120 & \text{\#10 bars or smaller} \\ 0.090 & \text{\#11 bars and larger} \end{cases}$$

The onset of strain hardening ε_{sh} shall be:

$$\varepsilon_{sh} = \begin{cases} 0.0150 & \text{\#8 bars} \\ 0.0125 & \text{\#9 bars} \\ 0.0115 & \text{\#10 \& \#11 bars} \\ 0.0075 & \text{\#14 bars} \\ 0.0050 & \text{\#18 bars} \end{cases}$$

A reduced ε^R_{su} equal to 0.06 shall be used for column longitudinal reinforcement.

$ksi := 1000 \dfrac{lbf}{in^2}$

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

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

$\varepsilon_{su} := 0.120$ for #8 bars

$\varepsilon_{sh} := 0.0150$ for #8 bars

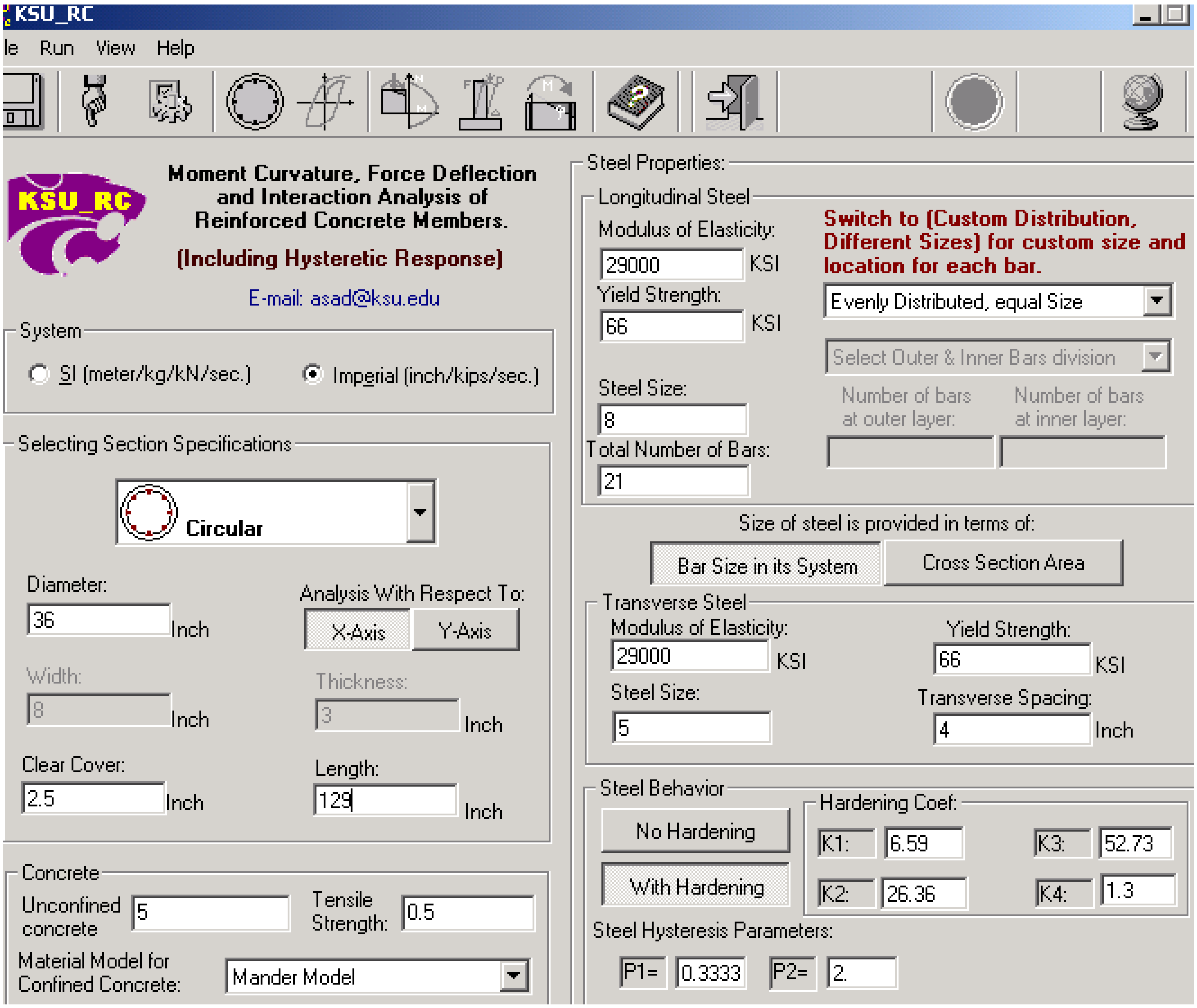
$\varepsilon R_{su} := 0.06$

$\varepsilon_y := \dfrac{f_{ye}}{29000ksi} \qquad \varepsilon_y = 0.002276$

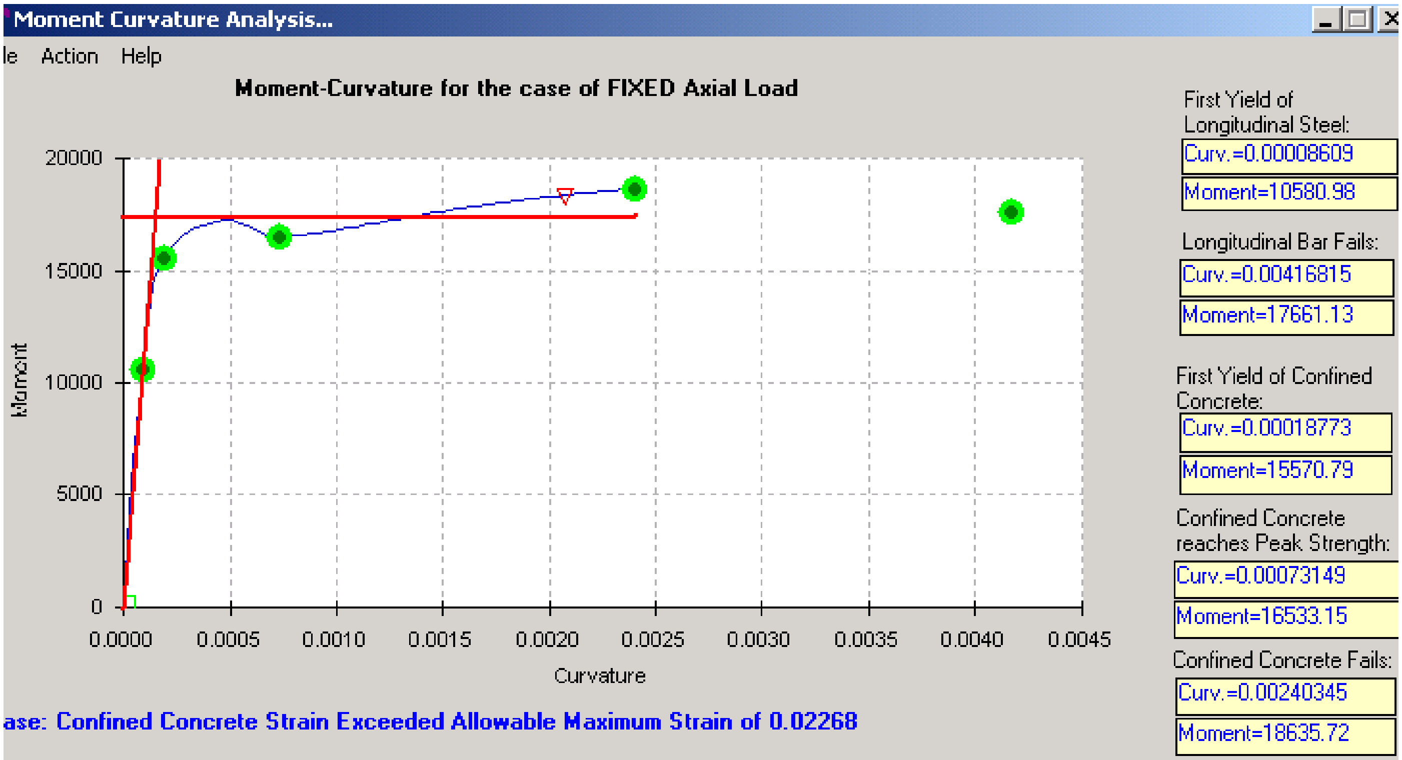
$K_1 := \dfrac{\varepsilon_{sh}}{\varepsilon_y} \qquad K_1 = 6.59 \qquad K_2 := \dfrac{\varepsilon R_{su}}{\varepsilon_y} \qquad K_2 = 26.36$

$K_3 := \dfrac{\varepsilon_{su}}{\varepsilon_y} \qquad K_3 = 52.73 \qquad L := \dfrac{H_o}{2} \qquad L = 129.54\,in$

This would be the input screen in KSU_RC for this analysis:



The analysis yields this M-ϕ curve:



From diagram above, Take M_p as 17500 in*kip $\quad M_p := 17500 \text{in} \cdot \text{kip} \quad M_p = 1458.33 \text{kip} \cdot \text{ft}$

COLUMN CQC FORCES (CONTINUED)

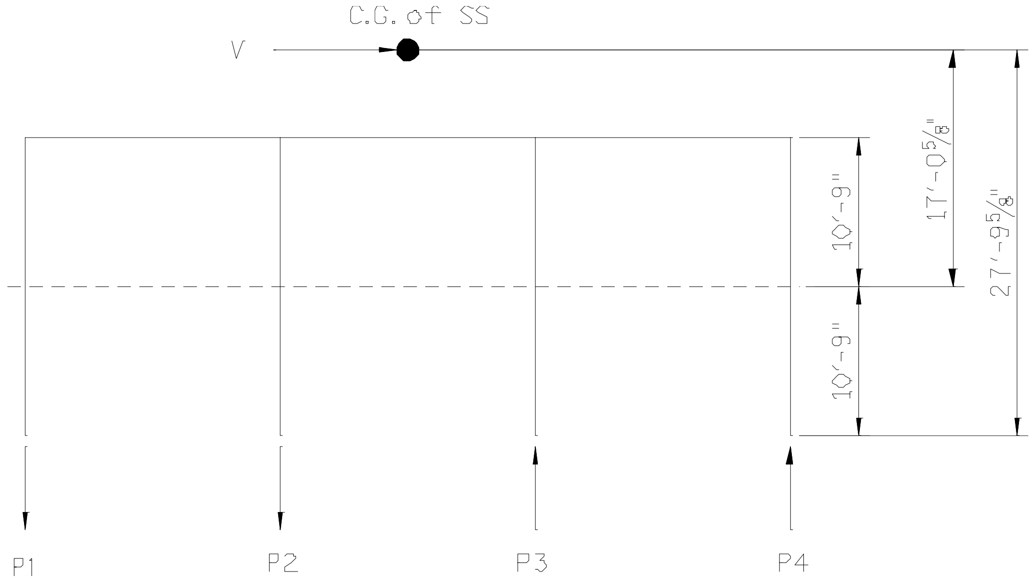
CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
3	BOT	1	17.6	434.	35.4	441.	14.3	2.4
		2	4.0	88.	167.9	2090.	61.5	13.2
		3	18.8	461.	85.8	1068.	32.7	6.4
		4	9.3	218.	178.6	2223.	65.7	13.9

Note that the elastic moment (from Seisab output) in the column exceeded the calculated plastic moment. It can be assumed that the column will form a plastic hinge.

$P_{dl} \cdot \Delta_{rL} = 11.275 \text{kip} \cdot \text{ft} \quad P_{dl} \cdot \Delta_{rT} = 11.275 \text{kip} \cdot \text{ft} \quad 0.25 \cdot M_p = 364.583 \text{kip} \cdot \text{ft}$

Structure meets P - Δ capacity requirements for SDC C.

If this was a bridge in SDC D, a pushover analysis would be required:



$EL_{CGSS} := 313.802ft$ $\quad EL_{TOF} := 286ft$ $\quad H := EL_{CGSS} - EL_{TOF} - \frac{21.5ft}{2}$ $\quad H = 17.052\,ft$

$V := 467.1kip$ Bent CQC from Seisab $\quad Col_{spa} := 18ft + 4in$

Note: H is taken to be length of the column from point of maximum moment to point of contraflexure - assumed to half the column height in the transverse direction plus the distance to the C.G. of the superstructure.

$Sum_d := 2\cdot\left[\left(0.5\cdot Col_{spa}\right)^2 + \left(1.5\cdot Col_{spa}\right)^2\right]$ $\quad Sum_d = 1680.56\,ft^2$

$P_1 := V\cdot H\cdot\left(\frac{1.5\cdot Col_{spa}}{Sum_d}\right)$ $\quad P_1 = 130.336\,kip$ $\quad P_4 := -P_1$

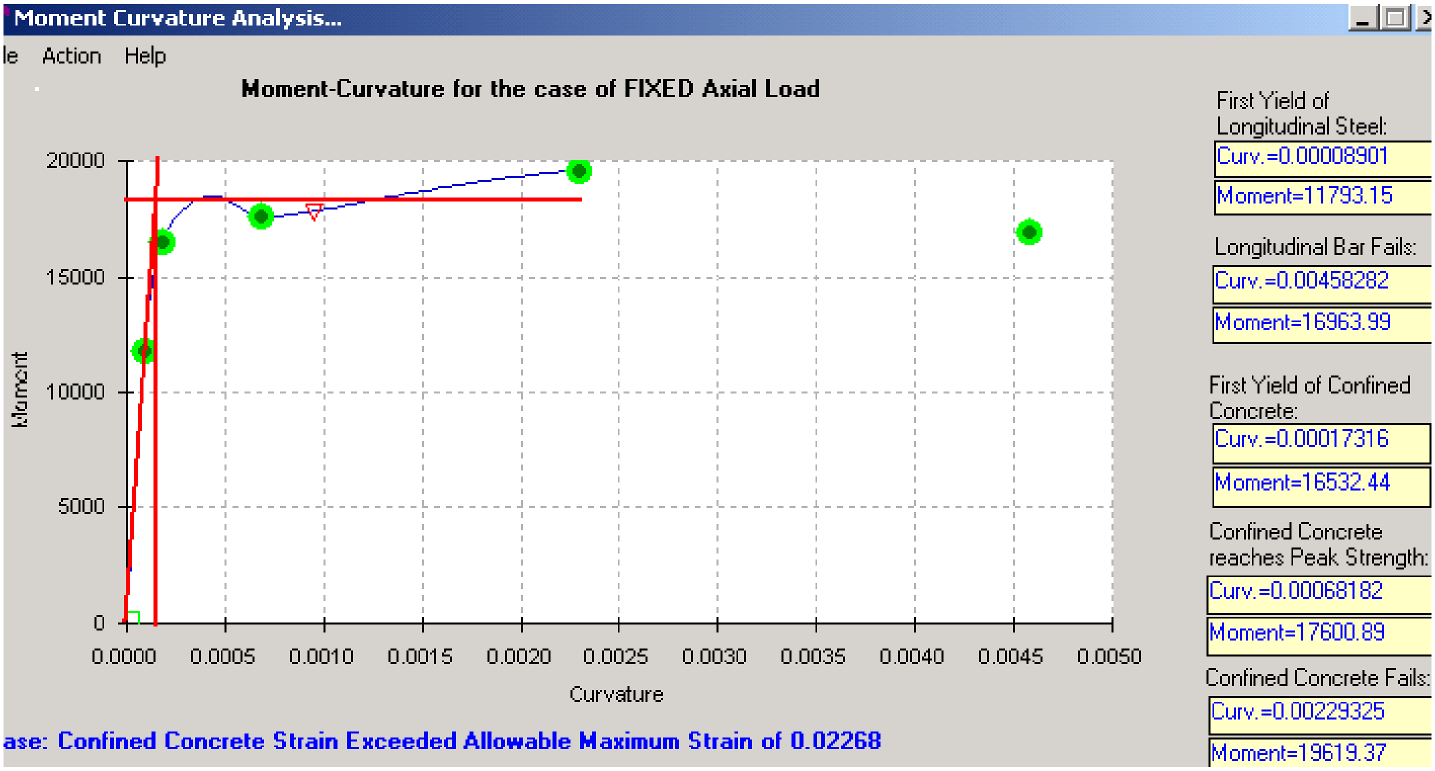
$P_2 := V\cdot H\cdot\left(\frac{0.5\cdot Col_{spa}}{Sum_d}\right)$ $\quad P_2 = 43.445\,kip$ $\quad P_3 := -P_2$

Check:

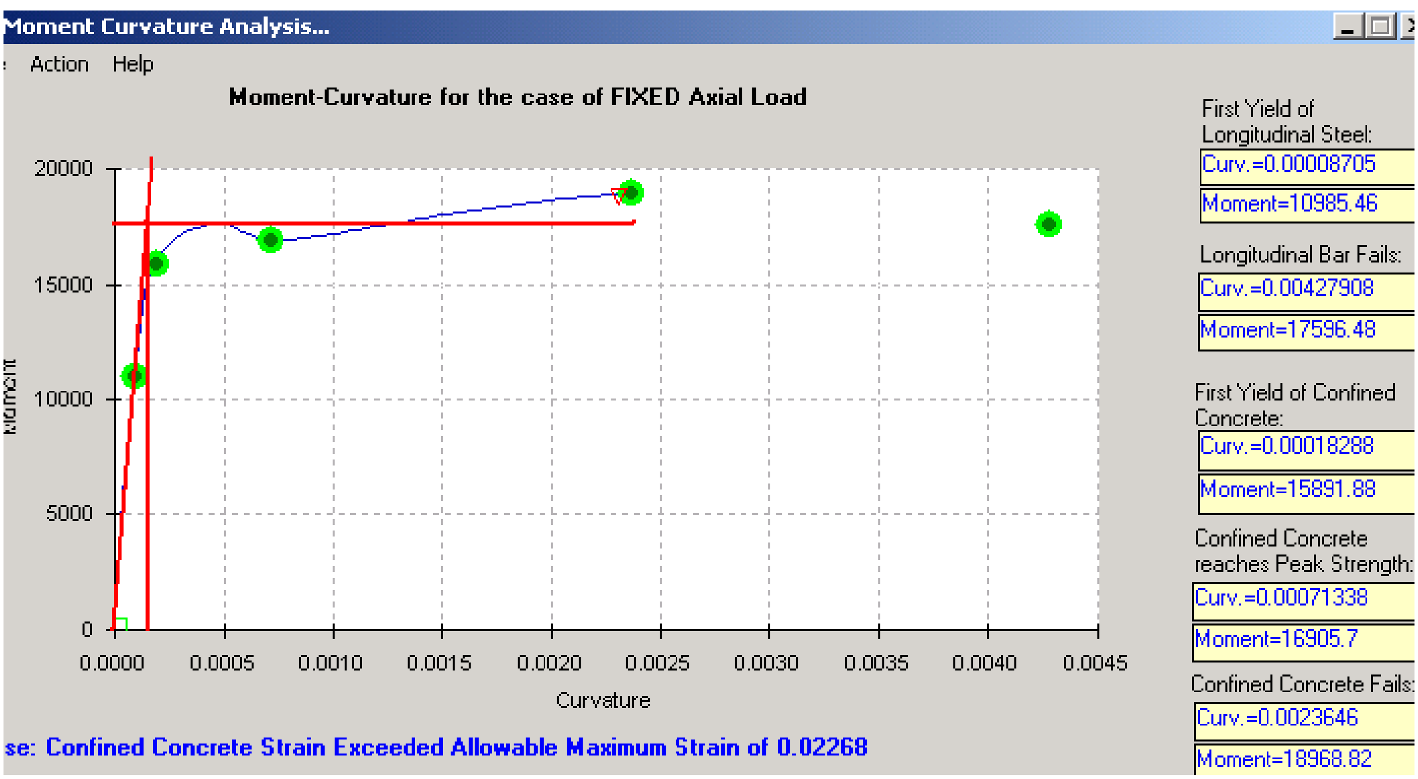
$P_1\cdot 1.5\cdot Col_{spa} + P_2\cdot 0.5\cdot Col_{spa} - P_3\cdot 0.5\cdot Col_{spa} - P_4\cdot 1.5\cdot Col_{spa} = 7964.99\,kip\cdot ft$

$V\cdot H = 7964.99\,kip\cdot ft$

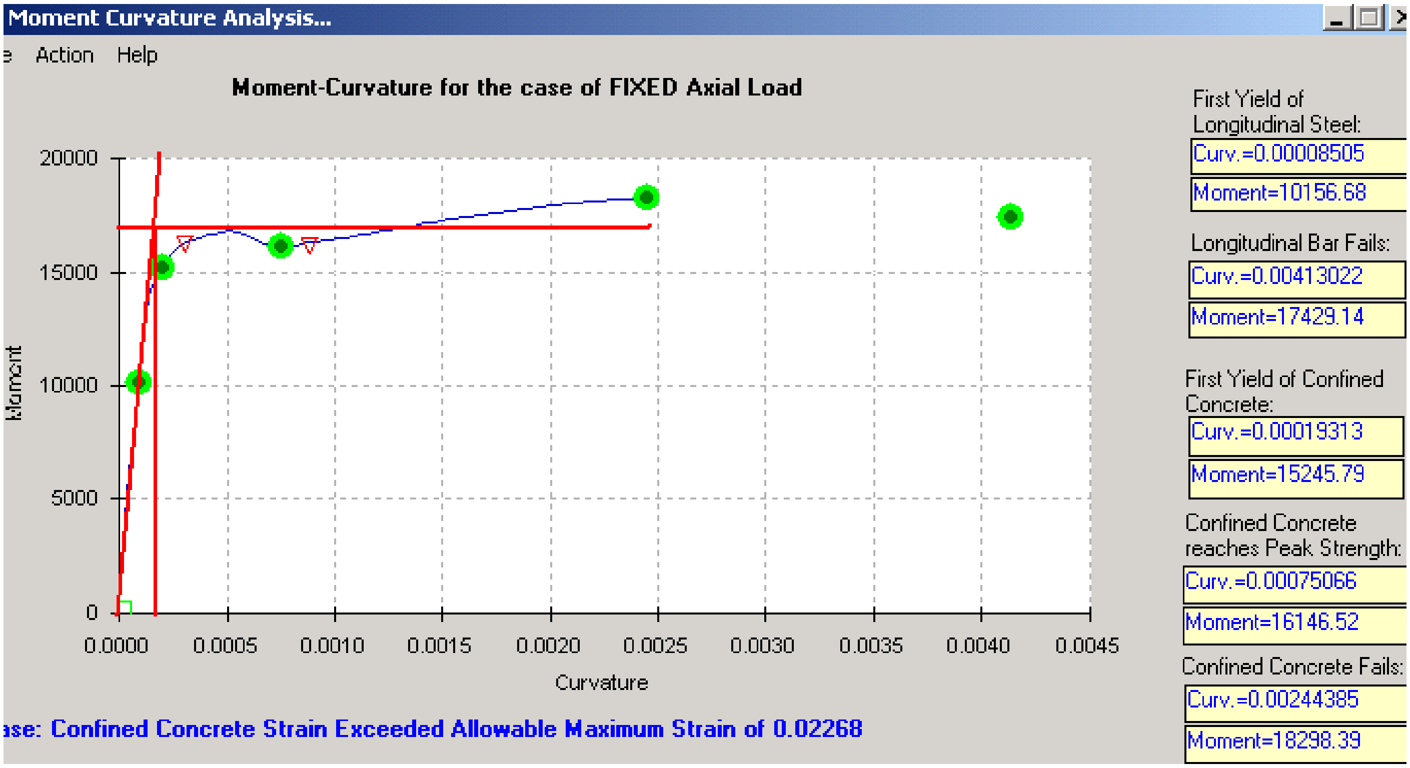
$P_1 + P_{dl} = 370.221\,kip$ $\quad P_2 + P_{dl} = 283.33\,kip$ $\quad P_3 + P_{dl} = 196.44\,kip$ $\quad P_4 + P_{dl} = 109.549\,kip$



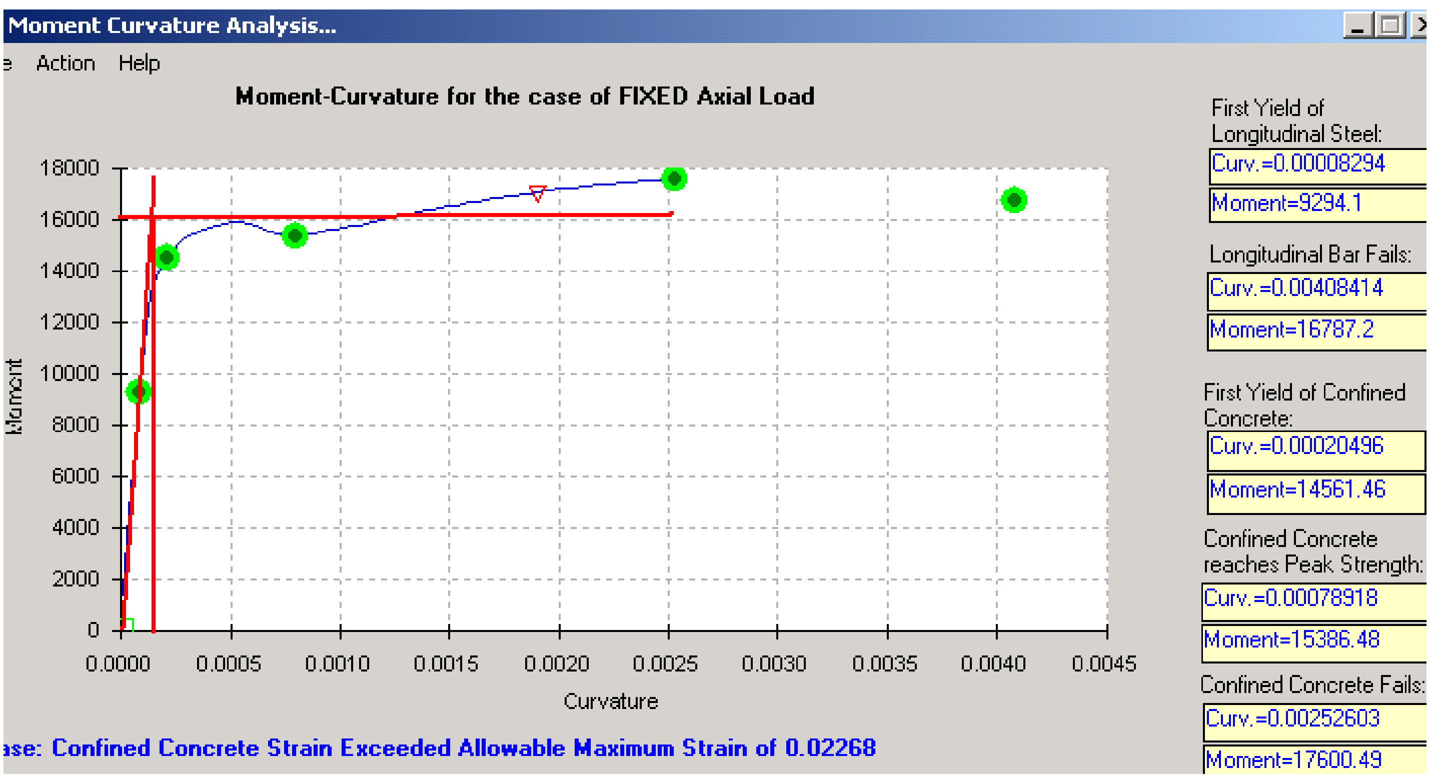
Take Mp = 1533 kip*ft for P = 370.22 kip $\quad M_{p1} := 1533 kip \cdot ft \quad \phi_{u1} := \frac{0.02293}{in} \quad \phi_{y1} := \frac{0.00015}{in}$



Take Mp = 1466 kip*ft for P = 283.33 kip $\quad M_{p2} := 1466 kip \cdot ft \quad \phi_{u2} := \frac{0.022364}{in} \quad \phi_{y2} := \frac{0.000175}{in}$



Take Mp = 1416 kip*ft for P = 196.44 kip $\quad M_{p3} := 1416\text{kip}\cdot\text{ft} \quad \phi_{u3} := \frac{0.0024438}{\text{in}} \quad \phi_{y3} := \frac{0.00018}{\text{in}}$



Take Mp = 1340 kip*ft for P = 109.55 kip $\quad M_{p4} := 1340\text{kip}\cdot\text{ft} \quad \phi_{u4} := \frac{0.002523}{\text{in}} \quad \phi_{y4} := \frac{0.0002}{\text{in}}$

$EI_1 := \frac{M_{p1}}{\phi_{y1}}$ $\quad EI_1 = 851666.67\,kip \cdot ft^2$ $\quad K_1 := \frac{3 \cdot EI_1}{(10.75ft)^3}$ $\quad K_1 = 2056.67\,\frac{kip}{ft}$

$EI_2 := \frac{M_{p2}}{\phi_{y2}}$ $\quad EI_2 = 698095.24\,kip \cdot ft^2$ $\quad K_2 := 3 \cdot \frac{EI_2}{(10.75ft)^3}$ $\quad K_2 = 1685.82\,\frac{kip}{ft}$

$EI_3 := \frac{M_{p3}}{\phi_{y3}}$ $\quad EI_3 = 655555.56\,kip \cdot ft^2$ $\quad K_3 := \frac{3 \cdot EI_3}{(10.75ft)^3}$ $\quad K_3 = 1583.09\,\frac{kip}{ft}$

$EI_4 := \frac{M_{p4}}{\phi_{y4}}$ $\quad EI_4 = 558333.33\,kip \cdot ft^2$ $\quad K_4 := \frac{3EI_4}{(10.75ft)^3}$ $\quad K_4 = 1348.31\,\frac{kip}{ft}$

$V_{p1} := \frac{M_{p1}}{10.75ft}$ $\quad V_{p1} = 142.605\,kip$

$V_{p2} := \frac{M_{p2}}{10.75ft}$ $\quad V_{p2} = 136.372\,kip$

$V_{p3} := \frac{M_{p3}}{10.75ft}$ $\quad V_{p3} = 131.721\,kip$

$V_{p4} := \frac{M_{p4}}{10.75ft}$ $\quad V_{p4} = 124.651\,kip$

$V_{p1} + V_{p2} + V_{p3} + V_{p4} = 535.349\,kip$

Initial assumed lateral force @ C.G. SS = 467.1 kip - will have to re-evaluate with a higher assumed lateral force.

$V := 540kip$ New Assumed value $Col_{spa} := 18ft + 4in$

Note: H is taken to be length of the column from point of maximum moment to point of contraflexure - assumed to half the column height in the transverse direction plus the distance to the C.G. of the superstructure.

$Sum_d := 2 \cdot \left[\left(0.5 \cdot Col_{spa}\right)^2 + \left(1.5 \cdot Col_{spa}\right)^2 \right]$ $Sum_d = 1680.56\, ft^2$

$P_1 := V \cdot H \cdot \left(\frac{1.5 \cdot Col_{spa}}{Sum_d} \right)$ $P_1 = 150.678\, kip$ $P_4 := -P_1$

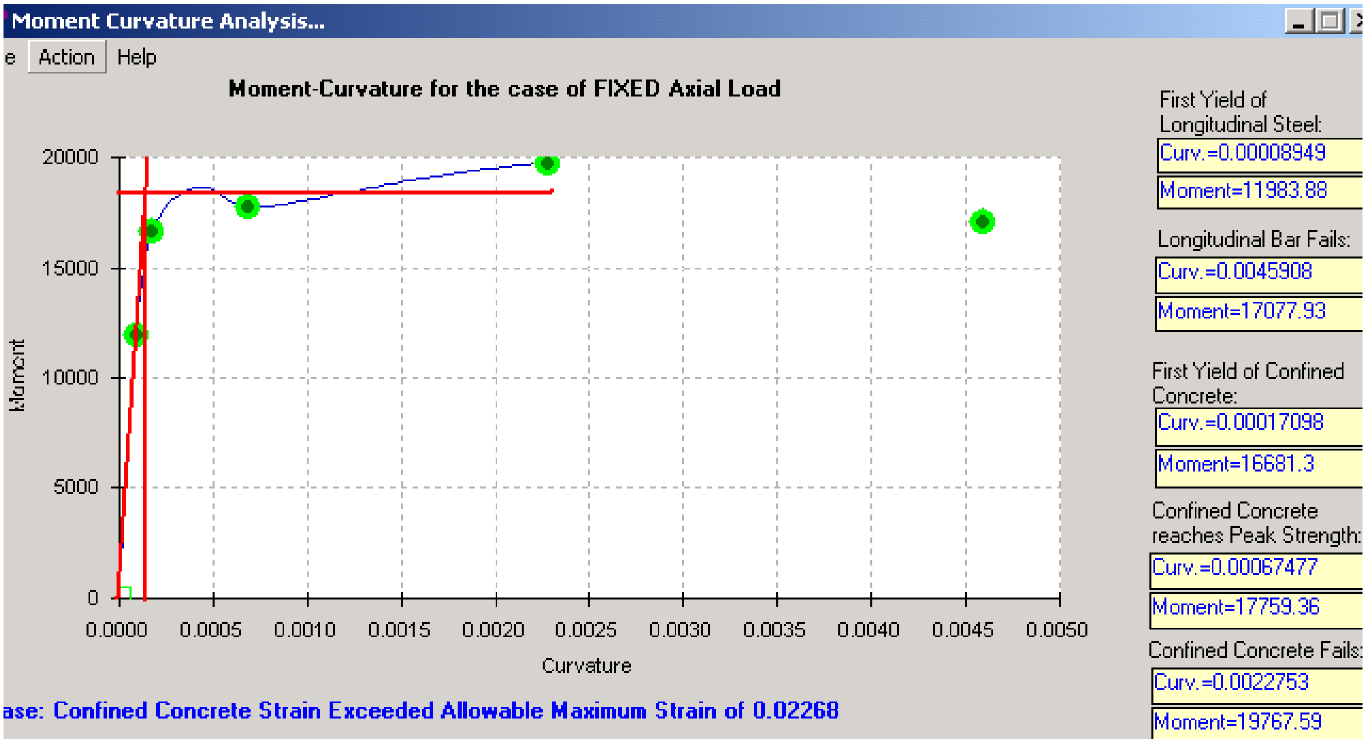
$P_2 := V \cdot H \cdot \left(\frac{0.5 \cdot Col_{spa}}{Sum_d} \right)$ $P_2 = 50.226\, kip$ $P_3 := -P_2$

Check:

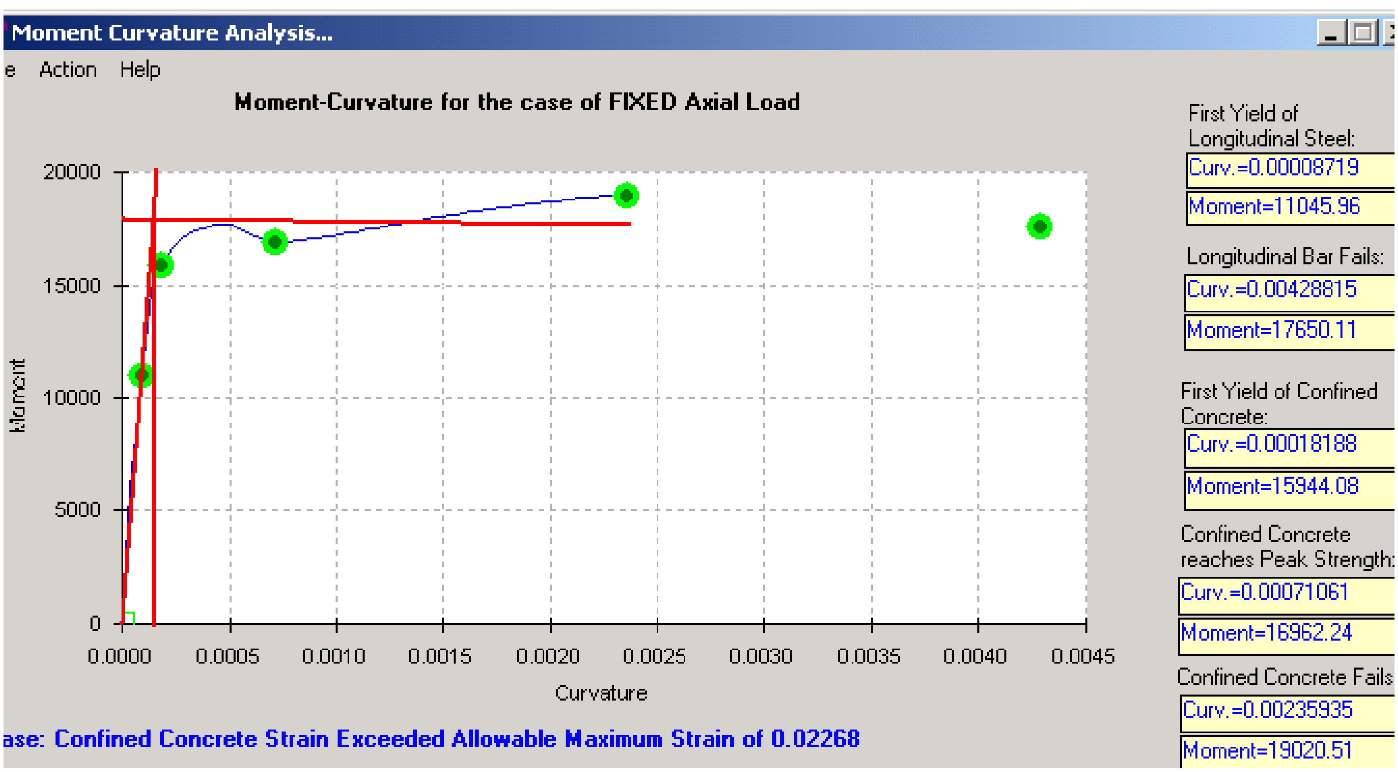
$P_1 \cdot 1.5 \cdot Col_{spa} + P_2 \cdot 0.5 \cdot Col_{spa} - P_3 \cdot 0.5 \cdot Col_{spa} - P_4 \cdot 1.5 \cdot Col_{spa} = 9208.08\, kip \cdot ft$

$V \cdot H = 9208.08\, kip \cdot ft$

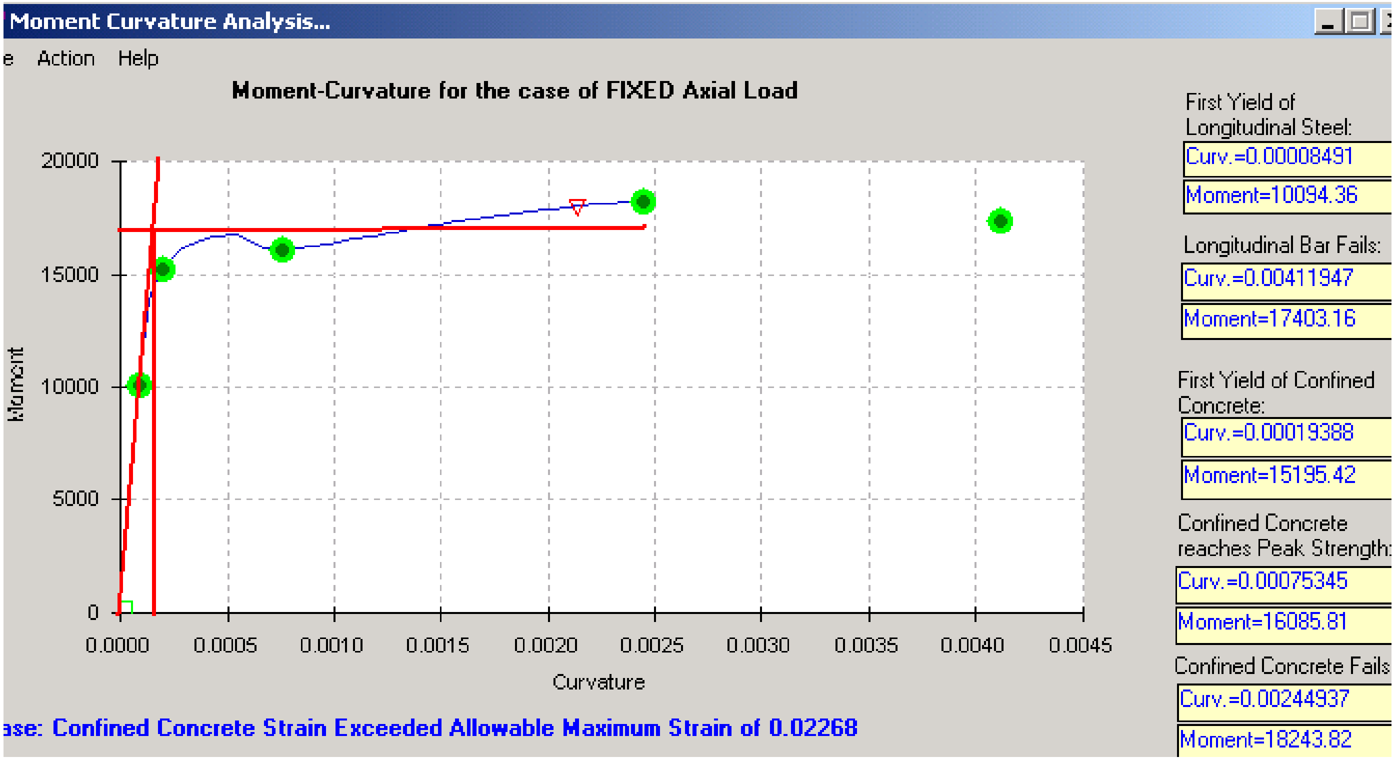
$P_1 + P_{dl} = 390.563\, kip$ $P_2 + P_{dl} = 290.111\, kip$ $P_3 + P_{dl} = 189.659\, kip$ $P_4 + P_{dl} = 89.207\, kip$



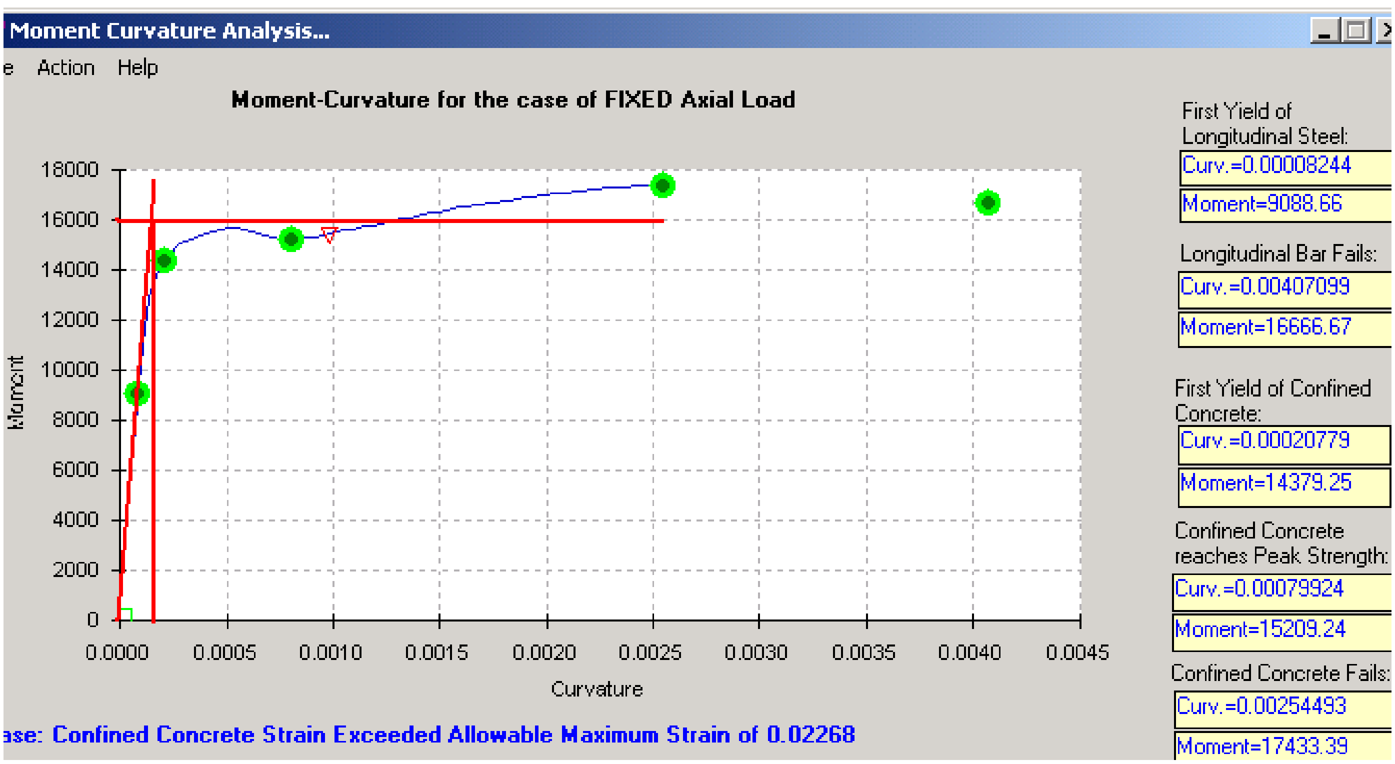
Take Mp = 1545 kip*ft for P = 390.56 kip $\qquad M_{p1} := 1545\text{kip}\cdot\text{ft} \qquad \phi_{u1} := \frac{0.002275}{\text{in}} \qquad \phi_{y1} := \frac{0.00014}{\text{in}}$



Take Mp = 1490 kip*ft for P = 290.11 kip $\qquad M_{p2} := 1490\text{kip}\cdot\text{ft} \qquad \phi_{u2} := \frac{0.002359}{\text{in}} \qquad \phi_{y2} := \frac{0.00015}{\text{in}}$



Take Mp = 1416 kip*ft for P = 189.66 kip $\quad M_{p3} := 1416 kip \cdot ft \quad \phi_{u3} := \frac{0.002449}{in} \quad \phi_{y3} := \frac{0.00016}{in}$



Take Mp = 1335 kip*ft for P = 89.21 kip $\quad M_{p4} := 1335 kip \cdot ft \quad \phi_{u4} := \frac{0.002545}{in} \quad \phi_{y4} := \frac{0.00017}{in}$

$EI_1 := \frac{M_{p1}}{\phi_{y1}}$ $\quad EI_1 = 919642.86\,kip \cdot ft^2$ $\quad K_1 := \frac{3 \cdot EI_1}{(10.75ft)^3}$ $\quad K_1 = 2220.83\,\frac{kip}{ft}$

$EI_2 := \frac{M_{p2}}{\phi_{y2}}$ $\quad EI_2 = 827777.78\,kip \cdot ft^2$ $\quad K_2 := 3 \cdot \frac{EI_2}{(10.75ft)^3}$ $\quad K_2 = 1998.99\,\frac{kip}{ft}$

$EI_3 := \frac{M_{p3}}{\phi_{y3}}$ $\quad EI_3 = 737500\,kip \cdot ft^2$ $\quad K_3 := \frac{3 \cdot EI_3}{(10.75ft)^3}$ $\quad K_3 = 1780.98\,\frac{kip}{ft}$

$EI_4 := \frac{M_{p4}}{\phi_{y4}}$ $\quad EI_4 = 654411.76\,kip \cdot ft^2$ $\quad K_4 := \frac{3EI_4}{(10.75ft)^3}$ $\quad K_4 = 1580.33\,\frac{kip}{ft}$

$V_{p1} := \frac{M_{p1}}{10.75ft}$ $\quad V_{p1} = 143.721\,kip$ $\quad \Delta_{yi1} := \frac{V_{p1}}{K_1}$ $\quad \Delta_{yi1} = 0.78\,in$

$V_{p2} := \frac{M_{p2}}{10.75ft}$ $\quad V_{p2} = 138.605\,kip$ $\quad \Delta_{yi2} := \frac{V_{p2}}{K_2}$ $\quad \Delta_{yi2} = 0.83\,in$

$V_{p3} := \frac{M_{p3}}{10.75ft}$ $\quad V_{p3} = 131.721\,kip$ $\quad \Delta_{yi3} := \frac{V_{p3}}{K_3}$ $\quad \Delta_{yi3} = 0.89\,in$

$V_{p4} := \frac{M_{p4}}{10.75ft}$ $\quad V_{p4} = 124.186\,kip$ $\quad \Delta_{yi4} := \frac{V_{p4}}{K_4}$ $\quad \Delta_{yi4} = 0.94\,in$

$V_{p1} + V_{p2} + V_{p3} + V_{p4} = 538.233\,kip$

Have acquired close to shear equilibrium - Assumed bent transverse force close to sum of plastic shears

Calculate Displacement Capacities:

Longitudinal direction:

$L := 21.5ft$ $\quad \Delta_y := \frac{0.00014}{in} \cdot \frac{L^2}{3}$ $\quad \Delta_y = 3.106\,in$

$$\Delta_p = (\phi_u - \phi_y) \cdot L_p \cdot \left(L - \frac{L_p}{2}\right)$$

4.11.6 Analytical Plastic Hinge Length

The analytical plastic hinge length, L_p, is the equivalent length of column over which the plastic curvature is assumed constant for estimating the plastic rotation. The plastic rotation is then used to calculate the plastic displacement of an equivalent member from the point of maximum moment to the point of contra-flexure. The plastic hinge lengths may be calculated for the two following conditions described below.

(a) Columns framing into a footing, an integral bent cap, an oversized shaft, or cased shaft:

$$L_p = 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} \quad \text{(in,ksi)} \quad (4.12)$$

$L_p := (0.08 \cdot 258 + 0.15 \cdot 66 \cdot 1)\text{in}$ $\quad L_p = 30.54\,\text{in}$

$\Delta_p := \left(\frac{0.002275}{\text{in}} - \frac{0.00014}{\text{in}}\right) \cdot L_p \cdot \left(L - \frac{L_p}{2}\right)$ $\quad \Delta_p = 15.827\,\text{in}$

$\Delta_u := \Delta_y + \Delta_p$ $\quad \Delta_u = 18.933\,\text{in}$ $\quad$ Compare this with 4.536" from implicit equation

Total Displacement Demand = 0.97"

4.9 MEMBER DUCTILITY REQUIREMENT FOR SDC D

Local member displacements such as column displacements, Δ_{col} are defined as the portion of global displacement attributed to the elastic displacement Δ_y and plastic displacement demand Δ_{pd} of an equivalent member from the point of maximum moment to the point of contra-flexure. Member section properties are obtained from a Moment-Curvature Analysis and used to calculate Δ_y and the plastic displacement capacity Δ_{pc}.

Local member ductility demand μ_D shall be computed based on the same equivalent member length as follows:

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_y} \quad (4.8)$$

For <u>conventional ductile design</u>, the local member ductility demand shall satisfy the following:

Single Column Bents	$\mu_D \leq 6$
Multi Column Bents	$\mu_D \leq 8$
Pier Walls Weak Ductile	$\mu_D \leq 6$
Pier Walls Strong Ductile	$\mu_D \leq 1$

Pile shafts are treated similar to columns.

Total plastic demand = 0, Δ_y > actual elastic demand

$\Delta_{pd} := 0\text{in}$

$\mu_D := 1 + \frac{\Delta_{pd}}{\Delta_y}$ $\quad \mu_D = 1 \quad < 8 \quad$ <u>O.K.</u>

Transverse Direction:

$L := 10.75ft \qquad \Delta_y := \frac{0.00014}{in} \cdot \frac{L^2}{3} \qquad \Delta_y = 0.777\,in$

$\Delta_p = (\phi_u - \phi_y) \cdot L_p \cdot \left(L - \frac{L_p}{2}\right) \qquad L_p := (0.08 \cdot 129 + 0.15 \cdot 66 \cdot 1)in \qquad L_p = 20.22\,in$

$\Delta_p := \left(\frac{0.002275}{in} - \frac{0.00014}{in}\right) \cdot L_p \cdot \left(L - \frac{L_p}{2}\right) \qquad \Delta_p = 15.827\,in \qquad \Delta_u := \Delta_y + \Delta_p \qquad \Delta_u = 16.603\,in$

Total Displacement demand = 1.451" $\quad \Delta_D := 1.451in \qquad \Delta_{pd} := \Delta_D - \Delta_y \qquad \Delta_{pd} = 0.674\,in$

$\mu_D := 1 + \frac{\Delta_{pd}}{\Delta_y} \qquad \mu_D = 1.868 \;\; < 8$ <u>O.K.</u>

Seismic Analysis of Pile Cap Footing:

6.4 PILE CAP FOUNDATION

6.4.1 General

The design of pile foundation for SDC B shall be based on forces determined by capacity design principles or elastic seismic forces, whichever is smaller.

The design of pile foundation for SDC C or D shall be based on forces determined by capacity design principles.

6.4.2 Foundation with Standard Size Piles

Standard size piles are considered to have a nominal dimension less than or equal to 16 inches.

Pile diameter = 14"

The provisions described below apply for columns with monolithic fixed connections to the footings designed for elastic forces as in SDC B or for column plastic hinge formation at the base as in SDC B, C, or D. For conformance to capacity design principles the foundations shall be designed to resist the overstrength column capacity M_o and the associated plastic shear V_o.

The design of standard size pile foundations in competent soil can be simplified using elastic analysis. For non-standard size piles, the distribution of forces to the piles and the pile cap may be influenced by the fixity of the pile connection to the pile cap in addition to the overall piles/pile cap flexibility. A more refined model that takes into account the pertinent parameters is recommended for establishing a more reliable force distribution.

A linear distribution of forces (see Figure 6.3) at different rows of piles, referred to as a simplified foundation model, is considered adequate provided a rigid footing response can be assumed. The rigid response of a footing can be assumed provided:

$$\frac{L_{ftg}}{D_{ftg}} \leq 2.5 \qquad (6.7)$$

where

L_{ftg} = The cantilever length of the pile cap measured from the face of the column to the edge of the footing.

D_{ftg} = The depth of the footing

Pile groups designed with the simplified foundation model can be sized to resist the plastic moment of the column M_p in lieu of M_{po} defined in Section 8.5.

For conforming to capacity design principles, the distribution of forces on these piles shall be examined about the X and Y axis in addition to the diagonal direction of the foundation cap.

The axial demand on an individual pile is found using Equations 6.8 and 6.9.

$$\left.\begin{matrix} C_{(i)}^{pile} \\ T_{(i)}^{pile} \end{matrix}\right\} = \frac{P_c}{N_p} \pm \frac{M_{p(y)}^{col} \times c_{x(i)}}{I_{p.g.(y)}} \pm \frac{M_{p(x)}^{col} \times c_{y(i)}}{I_{p.g.(x)}} \qquad (6.8)$$

$$I_{p.g.(y)} = \sum n \times c_{x(i)}^2 \qquad I_{p.g.(x)} = \sum n \times c_{y(i)}^2 \qquad (6.9)$$

where

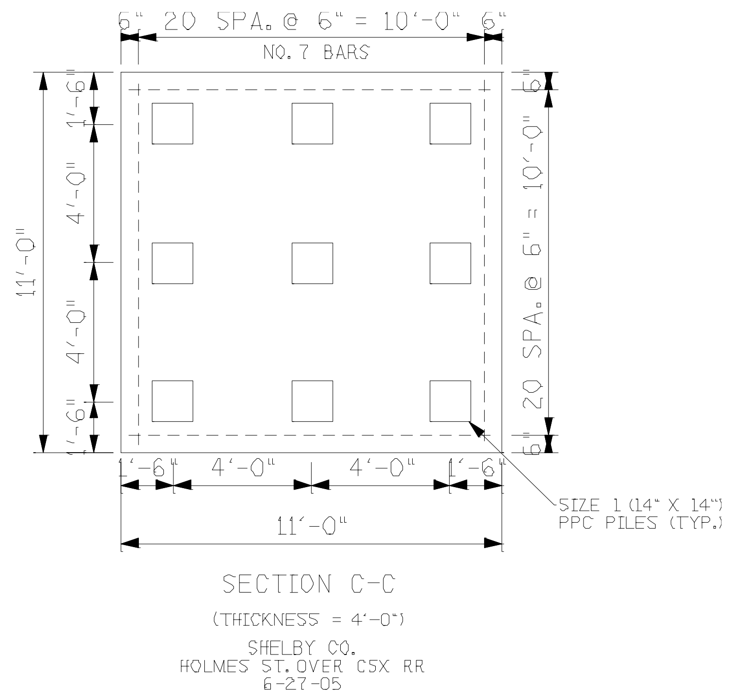
$I_{p.g.}$ = Moment of inertia of the pile group defined by Equation 6.9

$M_{p(y),(x)}^{col}$ = The component of the column plastic moment capacity about the X or Y axis

N_p = Total number of piles in the pile group

n = The total number of piles at distance $c_{x(i)}$ or $c_{y(i)}$ from the centroid of the pile group

P_c = The total axial load on the pile group including column axial load (dead load+EQ load), footing weight, and overburden soil weight



This is the footing that was designed with the previous seismic analysis from the 2004 LRFD specifications.

Analyze this pile cap footing and pile layout using the simplified foundation model and applying M_p in the transverse, X and Y axis:

$I_{pgx} := 6 \cdot (4ft)^2 \qquad I_{pgx} = 96\,ft^2$

$I_{pgy} := I_{pgx} \qquad I_{pgy} = 96\,ft^2$

$P_{footing} := (11ft)^2 \cdot 4ft \cdot 150\frac{lbf}{ft^3} \qquad P_{footing} = 72.6\,kip$

$P_{soil} := 5.5ft \cdot (11ft)^2 \cdot 120\frac{lbf}{ft^3} \qquad P_{soil} = 79.86\,kip$

from push-over analysis:

$P_c := 390.56kip + P_{footing} + P_{soil} \qquad P_c = 543.02\,kip$

$M_p := M_{p1} \qquad M_p = 1545\,kip \cdot ft$

6.4.4 Other Pile Requirements

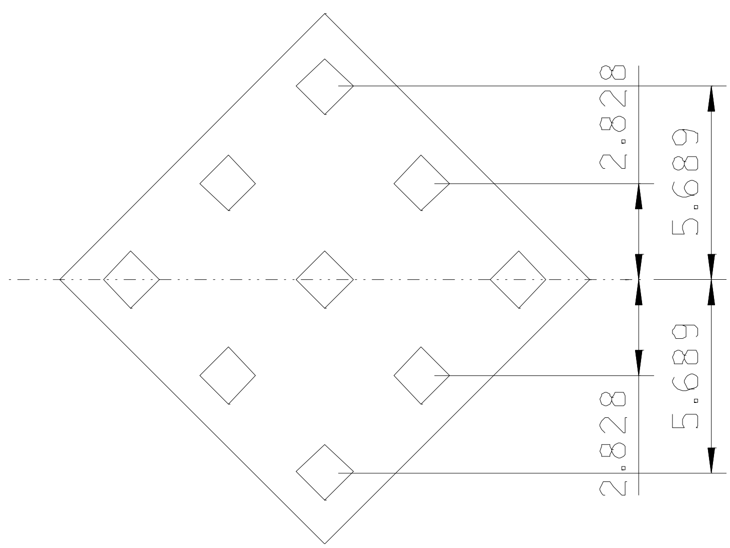
Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities, required to resist seismic loads shall be determined by means of the design criteria established in the site investigation report. Group reduction factors established in the geotechnical report shall be included in the analysis and design of piles required to resist lateral loads. The ultimate capacity of the piles should be used in designing for seismic loads.

When reliable uplift pile capacity from skin-friction is present, the pile/footing connection detail is present, and the pile/footing connection detail and structural capacity of the pile are adequate, uplifting of a pile footing is acceptable, provided that the magnitude of footing rotation will not result in unacceptable performance according to $P-\Delta$ requirements stated in Article 4.11.5. Friction piles may be considered to resist an intermittent but not sustained uplift. For seismic loads, tension resistance may be equivalent to 50 percent of the ultimate compressive axial load capacity. In no case shall the uplift exceed the weight of material (buoyancy considered) surrounding the embedded portion of the pile.

$$P_{pilemax} := \frac{P_c}{9} + \frac{M_p \cdot 4ft}{I_{pgx}} \qquad P_{pilemax} = 124.711\,kip$$

$P_{allowable} := 200kip$ (Ultimate pile capacity taken as 200kips)

Next, analyze the footing as if diagonal components of Mp were acting simultaneously in the footing about the 2 diagonal axis:



$$M_{paxis} := \frac{\sqrt{2}}{2} \cdot M_p \qquad M_{paxis} = 1092.48\,kip \cdot ft$$

$$I_{axis} := 2 \cdot (5.689ft)^2 + 4 \cdot (2.858ft)^2 \qquad I_{axis} = 97.402\,ft^2$$

$$P_{pilemaxdiagonal} := \frac{P_c}{9} + \frac{M_{paxis} \cdot 5.689ft}{I_{axis}} + \frac{M_{paxis} \cdot 5.689ft}{I_{axis}}$$

$P_{pilemaxdiagonal} = 187.953\,kip$ < 200 kip O.K.

Check uplift:

$$P_{maxuplift} := \frac{P_c}{9} - \frac{M_{paxis} \cdot 5.689ft}{I_{axis}} - \frac{M_{paxis} \cdot 5.689ft}{I_{axis}}$$

$P_{maxuplift} = -67.282\,kip$ < 100 kip O.K.

6.4.5 Footing Joint Shear SDC C and D

All footing/column moment resisting joints in SDC C and D shall be proportioned so the principal stresses meet the following criteria:

Principal compression:

$$p_c \le 0.25 f'_{ce} \quad (6.10)$$

Principal tension:

$$p_t \le 12\sqrt{f'_{ce}} \text{ (psi)} \quad (6.11)$$

Where:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \quad (6.12)$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \quad (6.13)$$

and

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} \times D_{ftg}} \quad (6.14)$$

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \quad (6.15)$$

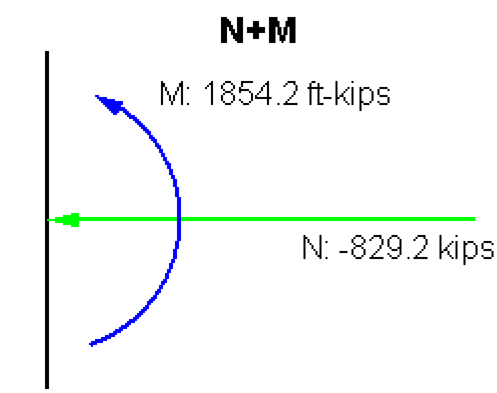
where

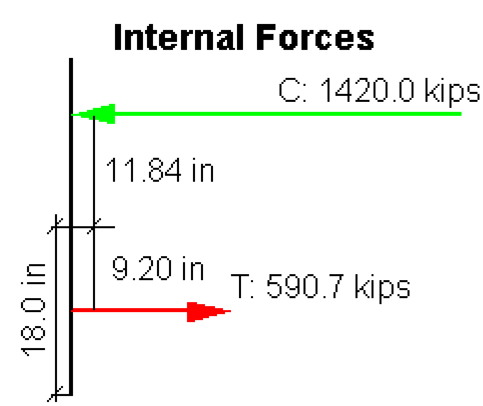
T_c = Column tensile force associated with M_o

From RC Pier output:

$M_o := 1.2 \cdot M_{p1}$ $M_o = 1854\,kip \cdot ft$

Using Response 2000, I iterated an input axial load until I obtained the Overstrength Plastic Moment. Response 2000 will solve for the internal tensile and compressive forces from this design moment:





$\sum T_{(i)}^{pile}$ = Summation of the hold down force in the tension piles.

$$B_{eff}^{ftg} = \begin{cases} \sqrt{2} x D_{cj} & \text{Circular Column} \\ B_c + D_{cj} & \text{Rectangular Column} \end{cases} \qquad (6.16)$$

$$f_v = \frac{P_{col}}{A_{jh}^{ftg}} \qquad (6.17)$$

P_{col} = Column axial force including the effects of overturning

$$A_{jh}^{ftg} = \begin{cases} (D_c + D_{ftg})^2 & \text{for Circular Column} \\ \left(B_c + \frac{D_{ftg}}{2}\right) x \left(D_c + \frac{D_{ftg}}{2}\right) & \text{for Rectangular Column} \end{cases} \qquad (6.18)$$

Where: A_{jh}^{ftg} is the effective horizontal area at mid-depth of the footing, assuming a 45° spread away from the boundary of the column in all directions, see Figure 6.4.

$T_e := 591\text{kip}$

$$\frac{P_c}{9} - \frac{M_{paxis} \cdot 2.828\text{ft}}{I_{axis}} - \frac{M_{paxis} \cdot 2.828\text{ft}}{I_{axis}} = -3.103\,\text{kip}$$

$SumT_{pile} := P_{maxuplift} + 2 \cdot -3.103\text{kip}$

$SumT_{pile} = -73.488\,\text{kip}$

$T_{jv} := T_e - 73.488\text{kip} \qquad T_{jv} = 517.512\,\text{kip}$

$B_{ftgeff} := \sqrt{2} \cdot 36\text{in} \qquad B_{ftgeff} = 50.912\,\text{in}$

$A_{ftgjh} := (36\text{in} + 48\text{in})^2 \qquad A_{ftgjh} = 7056\,\text{in}^2$

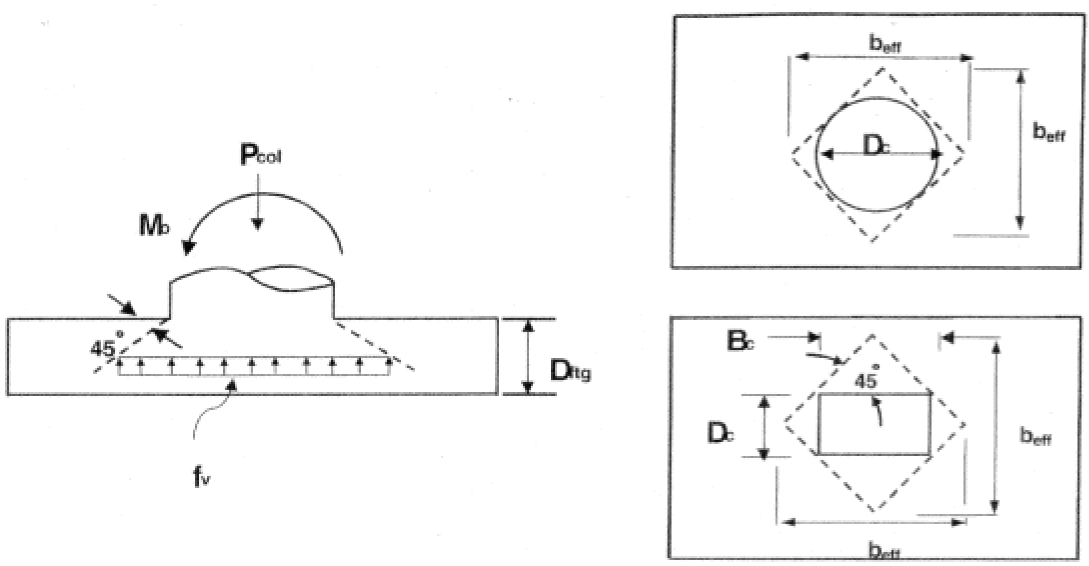


FIGURE 6.4: Effective Joint Width for Footing Joint Stress Calculation

$$v_{jv} := \frac{T_{jv}}{B_{ftgeff} \cdot 48\text{in}} \qquad v_{jv} = 211.769\,\frac{\text{lbf}}{\text{in}^2} \qquad P_{col} := 390.56\text{kip}$$ From pushover analysis

$$f_v := \frac{P_{col}}{A_{ftgjh}} \qquad f_v = 55.351\,\frac{\text{lbf}}{\text{in}^2} \qquad p_T := \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \qquad p_T = -26768.7\,\frac{\text{lbf}}{\text{ft}^2}$$

$p_c := \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^{\,2}}$ $\quad p_c = 241.245 \frac{lbf}{in^2}$ $\quad f'_{ce} := 5000 \frac{lbf}{in^2}$ $\quad \left(12 \cdot \sqrt{5000}\right)\frac{lbf}{in^2} = 848.528 \frac{lbf}{in^2}$

$P_c < f'_{ce}$ O.K. $\quad P_T < 848.53 \frac{lbf}{in^2}$ O.K.

5.2.3.3 *Abutment Stiffness and Passive Pressure Estimate*

Abutment stiffness, K_{eff}, and passive pressure capacity, P_p, should be characterized by a bi-linear or other higher-order nonlinear relationship as shown in Figure 5.3. Passive pressures may be assumed uniformly distributed over the height (H_w) of the backwall or diaphragm. Thus the total passive pressure force is:

$$P_p = p_p H_w \qquad (5.1)$$

where

H_w = wall height

p_p = passive pressure behind backwall per linear foot of wall unit length along the wall

If presumptive passive pressures are to be used for design, then the following criteria shall apply:

- Soil in the "passive pressure zone" should be compacted to a dry density greater than 95% of the maximum per ASTM Standard Method D1557 or equivalent.
- For cohesionless, non-plastic backfill (fines content less than 30%), the passive pressure P_p may be assumed equal to 2H/3 ksf per foot of wall length.
- For cohesive backfill (clay fraction > 15%), the passive pressure P_p may be assumed equal to 5 ksf provided the estimated unconfined compressive strength is greater than 4 ksf.

$k_p := 6 \qquad \gamma_s := 125 \frac{lbf}{ft^3}$

$H_w := 9.75in + 36in + 3in + 36in$

$p_p := k_p \cdot \gamma_s \cdot H_w \qquad p_p = 5.297 \frac{kip}{ft^2}$

$P_p := p_p \cdot H_w$

$P_p = 37.409 \frac{kip}{ft}$

$K_{eff1} := \frac{P_p}{0.02 \cdot H_w} \qquad K_{eff1} = 22.07 \frac{\frac{kip}{in}}{ft}$

b. Calculation of Soil Stiffness

An equivalent linear secant stiffness, K_{eff}, is required for analyses. For integral or diaphragm type abutments, an initial secant stiffness (Figure 5.3) may be calculated as follows:

$$K_{eff1} = P_p/0.02H_w \qquad (5.2)$$

The soil stiffness I calculated by the suggested specification gave a stiffness considerably less than the value derived by our current method. I have opted to use the seisab results obtained from using the stiffness values at the abutment derived by our current method.

Compare to value for soil stiffness computed by current method:

$$K_s := 50\,\frac{\frac{kip}{in}}{ft} - \frac{(8ft - H_w)}{8ft}\cdot 50\,\frac{\frac{kip}{in}}{ft} \qquad K_s = 44.141\,\frac{\frac{kip}{in}}{ft}$$

Check seismic capacity of column:

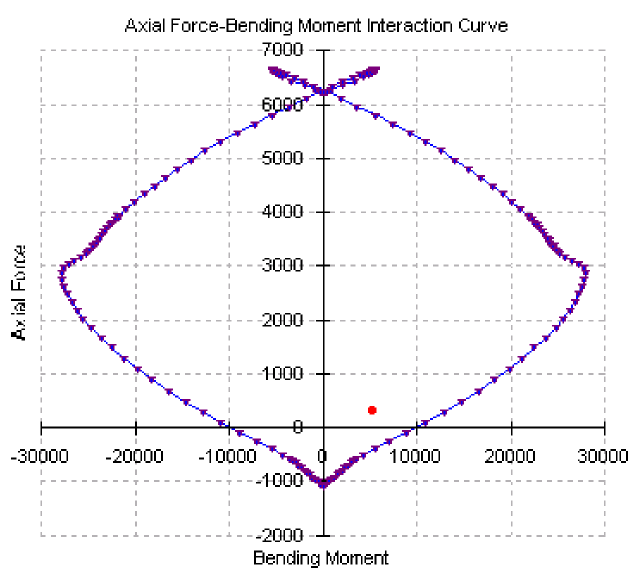
8.3 SEISMIC DESIGN CATEGORIES B, C, D

8.3.1 General

Initial sizing of columns can be performed using service load combinations. Columns may be governed by Strength Level Cases.

8.3.3 Force Demands SDC C & D

The design forces shall be based on forces resulting from plastic hinging or maximum connection capacity following capacity design principles as specified in Article 4.11. For SDC D where liquefaction is identified, plastic hinging in the foundation is acceptable as specified in Article 3.3.



From pushover analysis, these are the plastic hinging forces:

$P := 390.56kip$

$M_p = 1545\,kip\cdot ft$

From Seisab run, these are the elastic demand forces (modified with a factor of 5):

$$\frac{2096kip\cdot ft}{5} = 419.2\,kip\cdot ft$$

$$P_{dl} + \frac{167.5kip}{5} = 273.385\,kip$$

The elastic force demands will control.

As can be seen from the interaction diagram based on expected seismic material properties, the column will be adequate for seismic demands.

8.6.1 Shear Demand and Capacity

The shear demand for a column, V_d, in SDC B shall be determined based on the lesser of:

- The force obtained from an elastic linear analysis
- The force, V_o, corresponding to plastic hinging of the column including an overstrength factor

The shear demand for a column, V_d, in SDC C or D shall be determined based on the force, V_o, associated with the overstrength moment, M_{po}, defined in Article 8.5.

The column shear strength capacity shall be calculated based on the nominal material strength properties.

$$\phi V_n \geq V_d \qquad \phi = 0.85 \tag{8.9}$$

$$V_n = V_c + V_s \tag{8.10}$$

8.6.2 Concrete Shear Capacity SDC B, C and D

The concrete shear capacity of members designed for SDC B, C and D shall be determined as specified in Equation 8.11 through 8.24.

$$V_c = v_c A_e \tag{8.11}$$

$$A_e = 0.8 A_g \tag{8.12}$$

For members whose net axial load is in tension, $v_c = 0$

For shear stress capacity inside the plastic hinge zone

$$v_c = \alpha'\left(1 + \frac{P}{2000 A_g}\right)\sqrt{f'_{cc}} \leq 3.5\sqrt{f'_{cc}} \tag{8.13}$$

where,

P = axial compressive force (kips)

For columns with hoops or spirals

$$\alpha' = 0.015 \rho_s f_{yt} \quad \text{for SDC B} \tag{8.14}$$

$$\alpha' = 0.010 \rho_s f_{yt} \quad \text{for SDC C} \tag{8.15}$$

$$\alpha' = \frac{0.03}{\mu_D} \rho_s f_{yt} \quad \text{for SDC D} \tag{8.16}$$

$ksi := 1000 \frac{lbf}{in^2}$

$A_g := (18in)^2 \cdot \pi \qquad A_g = 1017.88\ in^2$

$V_{oT} := \frac{1.2 \cdot M_{p1}}{10.75ft} \qquad V_{oT} = 172.465\ kip$

$V_{oL} := \frac{1.2 \cdot M_{p1}}{21.5ft} \qquad V_{oL} = 86.233\ kip$

$V_o := \sqrt{V_{oT}^2 + V_{oL}^2} \qquad V_o = 192.822\ kip$

$V_d := V_o \qquad V_d = 192.822\ kip$

$\rho_s = \frac{A_{sp} \cdot \pi \cdot D_{sp}}{\left(\frac{\pi \cdot D_c^2}{4}\right) \cdot s} \qquad \rho_s = \frac{4 \cdot A_{sp} \cdot D_{sp}}{D_c^2 \cdot s}$

$cover := 2.5in \qquad D_{col} := 36in$

$D_c := D_{col} - 2 \cdot cover \qquad D_c = 31\ in$

$d_{spiral} := 0.625in \qquad f_{yt} := 60ksi$

$D_{sp} := D_{col} - 2 \cdot cover - d_{spiral} \qquad D_{sp} = 30.375\ in$

$s := 4in \qquad A_{sp} := 0.31in^2$

$\rho_s := \frac{4 \cdot A_{sp}}{D_{sp} \cdot s} \qquad \rho_s = 0.01021$

For SDC D the displacement ductility μ_D used to derive α' shall be based on the maximum local ductility demand in either of the principal local member axes.

For shear stress capacity outside the plastic hinge zone

$$v_c = \alpha''\left(1 + \frac{P}{2000A_g}\right)\sqrt{f'_{ce}} \le 3.5\sqrt{f'_{ce}} \quad (8.20)$$

For columns with spirals or hoops

$$\alpha'' = 0.03\rho_s f_{yt} \quad (8.21)$$

For tied reinforced columns

$$\alpha'' = 0.06\rho_w f_{yt} \quad (8.22)$$

Where the reinforcement ratio of spirals or hoops,

$$\rho_s = \frac{4A_{sp}}{Ds} \quad (8.23)$$

and the web reinforcement ratio (in the direction of bending)

$$\rho_w = \frac{A_v}{bs} \quad (8.24)$$

8.6.3 Shear Reinforcement Capacity

For confined circular or interlocking core sections, as described in Article 8.6.4, the shear reinforcement strength capacity is calculated using Equation 8.25.

$$V_s = \frac{\pi}{2}\left(\frac{A_v f_{yh} D}{s}\right), \quad (8.25)$$

where

8.6.5 Maximum Shear Reinforcement

The shear strength provided by the reinforcing steel, V_s, shall not be taken greater than:

$$8\sqrt{f'_{ce}}\ A_e\ (\text{psi}) \quad (8.28)$$

Inside the plastic hinge region:

$\alpha' := 0.010\cdot\rho_s\cdot 60000$ $\quad \alpha' = 6.123$

$P_{eq} := 167.5\text{kip}$ $\quad$ elastic

$P := P_{dl} - P_{eq}$ $\quad P = 72.385\ \text{kip}$

$$1 + \frac{72.385}{2\cdot 1017.88} = 1.036$$

$\alpha'\cdot 1.036 = 6.344$ $\ > 3.5$

$v_c = 3.5\cdot\sqrt{f'ce}$

$v_c := \left(3.5\cdot\sqrt{5000}\right)\text{psi}$ $\quad v_c = 0.247\ \text{ksi}$

$A_e := 0.8\cdot A_g$ $\quad A_e = 814.301\ \text{in}^2$

$V_c := v_c\cdot A_e$ $\quad V_c = 201.529\ \text{kip}$

$D := D_{sp}$

$$V_s := \frac{\pi}{2}\cdot\frac{0.31\text{in}^2\cdot 60\text{ksi}\cdot D}{4\text{in}}$$

$V_s = 221.865\ \text{kip}$ $\quad \left(8\cdot\sqrt{5000}\right)\text{psi}\cdot A_e = 460.638\ \text{kip}$

$\phi := 0.85$ $\quad V_n := V_s + V_c$ $\quad V_n = 423.394\ \text{kip}$

$\phi\cdot V_n = 359.885\ \text{kip}$ $\quad > 192.82\text{kip}$ O.K.

Outside the plastic hinge region:

$\rho_s := \dfrac{4\cdot A_{sp}}{D_{sp}\cdot 6\text{in}}$ $\quad \rho_s = 0.006804$ $\quad \alpha'' := 0.03\cdot\rho_s\cdot 60000$ $\quad \alpha'' = 12.247$ $\quad v_c := \left(3.5\cdot\sqrt{5000}\right)\text{psi}$

$v_c = 0.247\ \text{ksi}$ $\quad V_c = 201.529\ \text{kip}$ $\quad V_s := \dfrac{\pi}{2}\cdot\left(\dfrac{0.31\text{in}^2\cdot 60\text{ksi}\cdot D}{6\text{in}}\right)$ $\quad V_s = 147.91\ \text{kip}$

$\phi \cdot (V_s + V_c) = 297.023\,kip$ > 192.82 kip O.K.

8.6.6 Minimum Shear Reinforcement

The area of column spiral reinforcement, A_{sp}, or column web reinforcement A_v shall be determined based on Equations 8.23 and 8.24. The minimum spiral reinforcement ratio, ρ_s for each individual core of a column and the minimum web reinforcement ratio ρ_w shall be as follows:

For SDC B

$\rho_s = .2\%$

$\rho_w = .3\%$

For SDC C or D

$\rho_s = .4\%$

$\rho_w = .5\%$

$\rho_{smin} := 0.5\%$ $\rho_{smin} = 0.005$ < 0.006804 O.K.

8.7.2 Maximum Axial Load In A Ductile Member

The maximum axial load in a column, a pier wall, or a pile where ductility demand is greater than one shall not be greater than $0.2 f'_{ce} A_g$ where f'_{ce} is expected concrete strength and A_g is the gross cross-sectional area.

$0.2 \cdot f'_{ce} \cdot A_g = 1017.88\,kip$ > 390.56kip O.K.

8.8 LONGITUDINAL AND LATERAL REINFORCEMENT REQUIREMENTS

8.8.1 Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall not exceed the value specified in Equation 8.31.

$$0.04 \times A_g \quad (8.31)$$

$0.04 \cdot A_g = 40.715\,in^2$

$21 \cdot 0.79in^2 = 16.59\,in^2$ < 40.715 in^2 O.K.

8.8.2 Minimum Longitudinal Reinforcement

The minimum area of longitudinal reinforcement for compression members shall not be less than the value specified in Equations 8.32 thru 8.34.

$0.007 \times A_g$	for Columns in SDC B, C	(8.32a)
$0.01 \times A_g$	for Columns in SDC D	(8.32b)
$0.0025 \times A_g$	for Pier Walls in SDC B, C	(8.33)
$0.005 \times A_g$	for Pier Walls in SDC D	(8.34)

$0.007A_g = 7.125\,in^2$ For SDC C

$0.01 \cdot A_g = 10.179\,in^2$ For SDC D

Column with 21-#8 meets specification for both SDC C AND D.

8.8.3 Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDC C or D

Splicing of longitudinal column reinforcement in SDC C or D shall be outside the plastic hinging region as defined in Article 4.11.7. For SDC D ultimate strength splicing of reinforcement shall be used by means of mechanical couplers as Approved by Owner.

Currently, it is policy to locate the column splice in the center of the column for SDC C and D.

8.8.4 Minimum Development Length of Reinforcing Steel for SDC C or D

Column longitudinal reinforcement shall be extended into footings and cap beams as close as practically possible to the opposite face of the footing or cap beam.

The anchorage length for longitudinal column bars l_{ac} developed into the cap beam for seismic loads shall not be less than $24d_{bl}$ (in).

For SDC D, the anchorage length shall not be reduced by the addition of hooks or mechanical anchorage devices.

Our current policy is to extend the column steel into cap beams and footings a length equal to 1.25 l_d

For #8

$$l_d := \frac{0.63 \cdot 1\text{in} \cdot 60}{\sqrt{3}} \qquad l_d = 21.824\,\text{in} \qquad 1.25 \cdot l_d = 27.28\,\text{in}$$

$$24 \cdot 1\text{in} = 24\,\text{in}$$

The suggested specification is less stringent for development length, however currently it is allowed to use hooks to reduce development length for all design categories.

8.8.6 Maximum Bar Diameter for SDC C, or D

In order to ensure adequate bond to concrete, the nominal diameter of longitudinal reinforcement, $d_{b\ell}$, in columns shall satisfy Equation (8.35):

$$d_{b\ell} \leq \frac{25 \times \sqrt{f_c'}\,(L - 0.5D_C)}{f_{ye}} \quad \text{(in, psi)}, \quad (8.35)$$

where L is the length of column from the point of maximum moments to the point of contra-flexure established based on Capacity Design principles specified in Article 4.11. Where longitudinal bars in columns are bundled, this requirement of adequate bond shall be checked for the effective bar diameter, assumed as $1.2 \times d_{bb}$ for two-bar bundles, and $1.5 \times d_{bl}$ for three-bar bundles.

$$\frac{25 \cdot \sqrt{3000} \cdot (10.75\text{ft} - 0.5 \cdot 36\text{in})}{66000} = 2.303\,\text{in} > 1\text{in} \quad \underline{\text{O.K.}}$$

8.8.9 Maximum Spacing for Lateral Reinforcement for SDC C or D

The maximum spacing for lateral reinforcement in the plastic end regions shall not exceed the smallest of the following:

- One fifth of the least dimension of the cross-section for columns and one-half of the least cross-section dimension of piers.
- Six times the nominal diameter of the longitudinal reinforcement.
- 6 inches for single hoop or 8 inches for bundled hoops.

The lateral reinforcement shall extend in to the footing to the beginning of the longitudinal bar bend above the bottom mat. For the bent cap the longitudinal steel shall extend a distance to ensure adequate development length for the plastic hinge mechanism.

$$\frac{36in}{5} = 7.2\,in$$

$$6 \cdot 1in = 6\,in$$

8.12 SUPERSTRUCTURE DESIGN FOR NONINTEGRAL BENT CAP SDC C AND D

Nonintegral bent caps shall satisfy all requirements stated for frames with integral bent cap in the transverse direction. The minimum lateral transfer mechanism at the superstructure/substructure interface shall be established using an acceleration of 0.4g in addition to the overstrength capacity of shear keys or the elastic seismic force whichever is smaller.

Superstructure members supported on non-integral bent caps shall be simply supported at the bent cap or span continuously with a separation detail such as an elastomeric pad or isolation bearing between the bent cap and the superstructure. Refer to Type 3 choice of Article 7.2.

$R_{bent} := 752.7kip$ From RC Pier

$V_{obent} := 1.2 \cdot 540kip$ See pushover analysis

$V_{obent} = 648\,kip$

From Seisab output:

$$V_{elastic} := \sqrt{(130.6kip)^2 + (467.1kip)^2}$$

$$V_{elastic} = 485.014\,kip$$

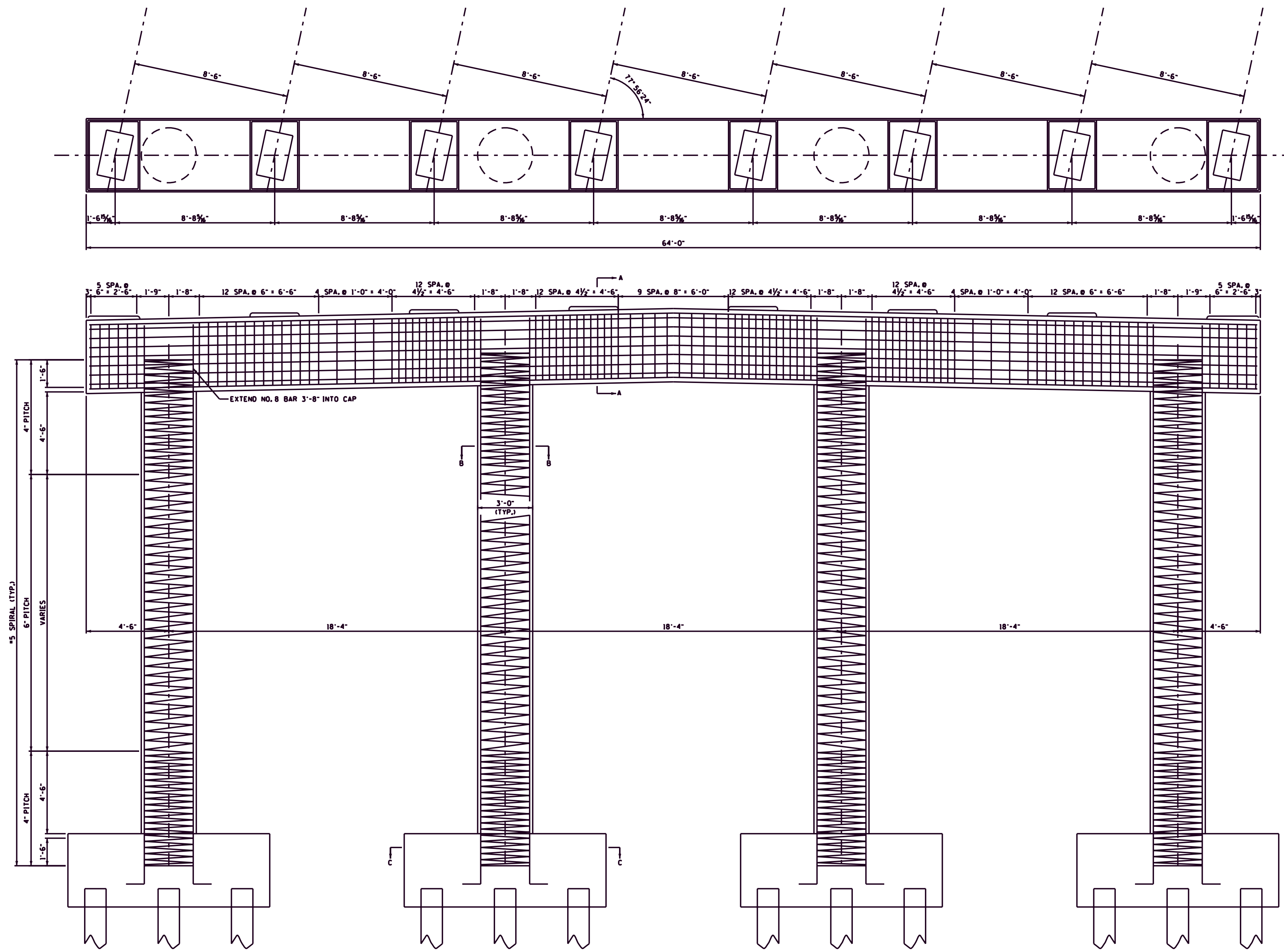
$$0.4 \cdot R_{bent} + V_{elastic} = 786.094\,kip$$

$No\_beams := 8$ $No\_bolts := 2$ $F_{ybolt} := 36ksi$ $Bolt\_area := 0.606in^2$

$F_{capacity} := No\_beams \cdot No\_bolts \cdot Bolt\_area \cdot F_{ybolt}$ $F_{capacity} = 349.056\,kip$

Bolts will need to be added to comply with this specification. Try adding 2 bolts in each bay (for a total of 30 bolts):

$F_{capacity} := 30 \cdot Bolt\_area \cdot F_{ybolt}$ $F_{capacity} = 654.48\,kip$



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SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AAAAAAAAA  BBBBBBBBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSS        EE            III     SSS        AA     AA  BB    BBB
 SSSSSS    EEEEE         III      SSSSSS    AAAAAAAAA  BBBBBBBB
  SSSSSS   EEEEE         III       SSSSSS   AAAAAAAAA  BBBBBBBB
      SSS  EE            III           SSS  AA     AA  BB    BBB
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*                                                              *
*                                                              *
*           Windows (GUI) By:  CV-McBridge Software            *
*                                                              *
*                     www.CV-McBridge.com                      *
*                                                              *
*                                                              *
*                                                              *
*------------------------- Licensed To: -----------------------*
*                                                              *
*           Tennessee Department of Transportation             *
*                                                              *
*------------------------- Mar 27, 2003 -----------------------*
*                                                              *
*                                                              *
*                                                              *
*                 Written By:  Roy Imbsen                      *
*                              Jon Lea                         *
*                              Clark Verkler                   *
*                              James Gates                     *
*                                                              *
****************************************************************

Date:  07-JUL-06                                 Time:  08:38:54
```

Echo of this file:
P:\STRUC_DS\Region IV\IYH\PaulChambers\Shelby Co. Holmes St\Holmes 1.ssb

---

```
# WinSeisab Version 5.0.5

# Length_Unit ft
# Force_Unit  kip
# Time_Unit   sec

# Length_Prec     4
# Force_Prec      4
# Time_Prec       4
# Area_Prec       6
# Volume_Prec     4
# Inertia_Prec    4
# Moment_Prec     4
# LinWeight_Prec  4
# Stress_Prec     4
# Density_Prec    4
# Trans_K_Prec    4
# Rotat_K_Prec    4
# Couple_K_Prec   8
# Accel_Prec      2

# ShowExponentialK   No


SEISAB "Shelby Co. Holmes St. over CSX RR"
   RESPONSE SPECTRUM ANALYSIS
   SUPERSTRUCTURE JOINTS   3
   COLUMN JOINTS   2
   OUTPUT LEVEL   0
   BLOCKING FACTOR   0



ALIGNMENT
   STATION   0.0
   COORDINATES   N  0.0   E  0.0
   BEARING   N 3 51 46 E



SPANS
   LENGTHS         40.0  60.0  48.0
   AREA            65.875  65.875  65.875
   I11             9.604  9.604  9.604
   I22             24249.1  24249.1  24249.1
   I33             86.7021  86.7021  86.7021
   A22             0.0  0.0  0.0
   A33             0.0  0.0  0.0
   DENSITY         0.15  0.15  0.15
   WEIGHT          3.078  3.078  3.078
   E               478160.78  478160.78  478160.78
   PRATIO          0.2  0.2  0.2
```

```
DESCRIBE
   COLUMN   'Type 1'
      AREA            7.069
      I11             7.952
      I22             3.976
      I33             3.976
      A22             0.0
      A33             0.0
      DENSITY         0.15
      E               617303.0
      PRATIO          0.2



   SPECIAL CAP   'Type 1'
      I33       21.333
      A22       16.0
      E         478161.0
      PRATIO    0.2



   BEARING ELEMENT   'elasto'
      KF1F1   1000000000.0
      KF2F2   1000000000.0
      KF3F3   1000000000.0
      KM1M1   0.0
      KM2M2   0.0
      KM3M3   0.0



ABUTMENT STATION 0.0
   BEARING  N 81 48 10 E  N 81 48 10 E

   ELEVATION TOP 312.082 AT ABUTMENT 1
   ELEVATION TOP 313.812 AT ABUTMENT 4

   CONNECTION PIN    AT ABUTMENT 1
   CONNECTION PIN    AT ABUTMENT 4



BENT
   BEARING  N 81 48 10 E  N 81 48 10 E

   ELEVATION TOP       313.042  313.802
   ELEVATION BEARINGS  310.222  310.982
   ELEVATION CAP       308.097  308.857
   ELEVATION BOTTOM    284.0  284.0
   WEIGHT  192.4125  192.4125

   COLUMN  SKEWED LAYOUT  'Type 1'  18.333  'Type 1'  18.333  'Type 1' -
           18.333  'Type 1'  AT BENT 2
   COLUMN  SKEWED LAYOUT  'Type 1'  18.333  'Type 1'  18.333  'Type 1' -
           18.333  'Type 1'  AT BENT 3

   COLUMN TOP    FIX    AT BENT 2
   COLUMN BOTTOM FIX    AT BENT 2
```

```
   COLUMN TOP    FIX    AT BENT 3
   COLUMN BOTTOM FIX    AT BENT 3

   SPECIAL CAP 'Type 1'  AT BENT 2
   SPECIAL CAP 'Type 1'  AT BENT 3

   CONNECTION  BEARING ELEMENT SKEWED LAYOUT  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               AT BENT 2
   CONNECTION  BEARING ELEMENT SKEWED LAYOUT  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               AT BENT 3


FOUNDATION
   AT ABUTMENT  1 4
      SPRING CONSTANTS
         KF1F1   22119.5
C --     KF2F2   fixed
         KF3F3   3840.0
C --     KM1M1   fixed
C --     KM2M2   fixed
C --     KM3M3   fixed
      WEIGHT  0.0



LOADS
   USE FIXED NOTATION FOR VIBRATION INFO
   USE FIXED NOTATION FOR DISPLACEMENTS
   USE FIXED NOTATION FOR FORCES

   RESPONSE SPECTRUM
      COMBINATION FACTOR   0.3
C --  MODE SHAPES  -- using default
      DAMPING COEFFICIENT   0.05

      ARBITRARY CURVE
         PERIOD    0.0000  0.0860  0.4180  0.5000  0.6000  0.7000  0.8000 -
                   0.9000  1.0000  1.1000  1.2000  1.3000  1.4000  1.5000 -
                   2.0000  2.5000  3.0000  3.5000  4.0000  4.5000  5.0000
         VALUE     0.3260  0.8140  0.8140  0.6960  0.5800  0.5000  0.4350 -
                   0.3870  0.3480  0.3160  0.2900  0.2670  0.2490  0.2320 -
                   0.1740  0.1390  0.1160  0.0990  0.0870  0.0770  0.0700

      GRAVITY   32.2

FINISH
```

```
- - - - WinSEISAB - - - -        (Version 5.0.5)        07-JUL-06
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Shelby Co. Holmes St. over CSX RR
_________________________________________________________________

                      WinSEISAB DATA INPUT

-----------------------------------------------------------------


      ****************************************************
      *                                                  *
      *                                                  *
      *                                                  *
      *                                                  *
      *                RESPONSE SPECTRUM                 *
      *                                                  *
      *                                                  *
      *                                                  *
      *                                                  *
      ****************************************************




           THE BRIDGE MODEL GENERATED IS BASED ON:

                3 INTERMEDIATE JOINT(S) ON EACH SPAN

                2 INTERMEDIATE JOINT(S) ON EACH COLUMN
```

Shelby Co. Holmes St. over CSX RR

ALIGNMENT DATA INPUT

INITIAL REFERENCE POINT AND ALIGNMENT

STATION	STATION COORDINATE	OFFSET DIR	OFFSET VALUE	BEARING
0.00	N 0.00 E 0.00		0.00	N 3 51 46 E

Shelby Co. Holmes St. over CSX RR

SPAN DATA INPUT

SEGMENT LENGTHS / SECTION AND MATERIAL PROPERTIES

SEGMENT	LENGTH	AREA	I11	I22	I33
SPAN 1	40.00				
1	40.00	65.87	9.60	24249.10	86.70
SPAN 2	60.00				
1	60.00	65.87	9.60	24249.10	86.70
SPAN 3	48.00				
1	48.00	65.87	9.60	24249.10	86.70

SEGMENT	A22	A33	ELASTIC MODULUS	P- RATIO	DENSITY	WT/LENGTH
SPAN 1						
1	0.0	0.0	478161.	0.20	0.150	3.078
SPAN 2						
1	0.0	0.0	478161.	0.20	0.150	3.078
SPAN 3						
1	0.0	0.0	478161.	0.20	0.150	3.078

- - - - WinSEISAB - - - - (Version 5.0.5) 07-JUL-06
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Shelby Co. Holmes St. over CSX RR

DESCRIBE DATA INPUT

COLUMN INFORMATION

NO. 1 TITLE: Type 1
INFORMATION:

SEGMENT	LENGTH	AREA	I11	I22	I33
1	0.00	7.07	7.95	3.98	3.98

SEGMENT	A22	A33	ELASTIC MODULUS	P-RATIO	DENSITY
1	0.0	0.0	617303.	0.20	0.15

SPECIAL BENT CAP INFORMATION

NO. 1 TITLE: Type 1
INFORMATION:
DENSITY:
ELASTIC MODULUS: 478161.
POISSONS RATIO: 0.20

	AREA	I11	I22	I33	A22	A33
SPECIAL CAP MEMBER				21.33	16.0	

DESCRIBE DATA INPUT (CONTINUED)

BEARING ELEMENT INFORMATION

NO. 1 TITLE: elasto
INFORMATION:

KF1F1	KF1F2	KF1F3	KF1M1	KF1M2	KF1M3
1.0E+09	0.	0.	0.	0.	0.

KF2F2	KF2F3	KF2M1	KF2M2	KF2M3
1.0E+09	0.	0.	0.	0.

KF3F3	KF3M1	KF3M2	KF3M3
1.0E+09	0.	0.	0.

KM1M1	KM1M2	KM1M3
0.	0.	0.

KM2M2	KM2M3
0.	0.

KM3M3
0.

Shelby Co. Holmes St. over CSX RR

ABUTMENT DATA INPUT

GEOMETRY AND GENERAL INFORMATION

ABT	STATION	BEARING	SUPER CG ELEVATION	WALL BOT ELEVATION	ABUTMENT CONNECTION
1	0.00	N 81 48 10 E	312.08		PIN
4		N 81 48 10 E	313.81		PIN

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Shelby Co. Holmes St. over CSX RR

BENT DATA INPUT

BENT GEOMETRY, WEIGHT AND CAP INFORMATION

BENT	BEARING	WEIGHT	CAP TITLE	NOTE
2	N 81 48 10 E	192.41	Type 1	
3	N 81 48 10 E	192.41	Type 1	

BENT ELEVATION INFORMATION

BENT	SUPER CG ELEVATION	DIAPHRAGM ELEVATION	BEAR. ELMT ELEVATION	BENT CAP ELEVATION	FOOTING ELEVATION
2	313.04		310.22	308.10	284.00
3	313.80		310.98	308.86	284.00

BENT CONNECTIVITY INFORMATION

BENT	SUPERSTR CONTINUITY	BENT TO SUPERSTR CONNECTION TYPE
2	CONTINUOUS	BEARING
3	CONTINUOUS	BEARING

BENT DATA INPUT (CONTINUED)

BENT BEARING ELEMENT LAYOUT

BENT	LOCATION	SPACING	BE	TITLE	DISTANCE
2		SKEWED	1	elasto	8.69
			2	elasto	8.69
			3	elasto	8.69
			4	elasto	8.69
			5	elasto	8.69
			6	elasto	8.69
			7	elasto	8.69
			8	elasto	
3		SKEWED	1	elasto	8.69
			2	elasto	8.69
			3	elasto	8.69
			4	elasto	8.69
			5	elasto	8.69
			6	elasto	8.69
			7	elasto	8.69
			8	elasto	

BENT COLUMN LAYOUT

					....GROUP	OFFSET...	
BENT	SPACING	COL	TITLE	DISTANCE	SPCNG	DIR	OFFSET
2	SKEWED	1	Type 1	18.33			0.00
		2	Type 1	18.33			
		3	Type 1	18.33			
		4	Type 1				
3	SKEWED	1	Type 1	18.33			0.00
		2	Type 1	18.33			
		3	Type 1	18.33			
		4	Type 1				

BENT DATA INPUT (CONTINUED)

BENT COLUMN END INFORMATION

		............TOP...........			..........BOTTOM..........		
BNT	COL	LT LT ASSTMM	BRG ELMT	JNT SIZE	LT LT ASSTMM	BRG ELMT	JNT SIZE
2	1			0.00			0.00
	2			0.00			0.00
	3			0.00			0.00
	4			0.00			0.00
3	1			0.00			0.00
	2			0.00			0.00
	3			0.00			0.00
	4			0.00			0.00

FOUNDATION DATA

ABUTMENT FOUNDATION INFORMATION

ABUTMENT	WEIGHT	ROT. ANGLE	FOUNDATION TYPE	FTNG TITLE
1	0.00	0.00	SPRING CONSTANTS	
4	0.00	0.00	SPRING CONSTANTS	

ABUTMENT FOUNDATION SPRING CONSTANTS

ABUT	KF1F1	KF1F2	KF1F3	KF1M1	KF1M2	KF1M3
1	22120.		0.			

KF2F2	KF2F3	KF2M1	KF2M2	KF2M3
DOF FIXD				

KF3F3	KF3M1	KF3M2	KF3M3
3840.			

KM1M1	KM1M2	KM1M3
DOF FIXD		

KM2M2	KM2M3
DOF FIXD	

KM3M3
DOF FIXD

FOUNDATION DATA (CONTINUED)

ABUTMENT FOUNDATION SPRING CONSTANTS (CONTINUED)

```
ABUT         KF1F1    KF1F2    KF1F3    KF1M1    KF1M2    KF1M3
----       -------- -------- -------- -------- -------- --------
  4          22120.                0.

                      KF2F2    KF2F3    KF2M1    KF2M2    KF2M3
                    -------- -------- -------- -------- --------
                    DOF FIXD

                               KF3F3    KF3M1    KF3M2    KF3M3
                             -------- -------- -------- --------
                                3840.

                                        KM1M1    KM1M2    KM1M3
                                      -------- -------- --------
                                      DOF FIXD

                                                 KM2M2    KM2M3
                                               -------- --------
                                               DOF FIXD

                                                          KM3M3
                                                        --------
                                                        DOF FIXD
```

LOADINGS DATA INPUT

THE NUMBER OF MODE SHAPES FOUND WILL BE 9

ACCELERATION SPECTRUM INFORMATION

USER SPECIFIED ACCELERATION SPECTRUM

DAMPING COEFFICIENT = 0.05
ACCELERATION DUE TO GRAVITY = 32.20

DIGITIZED ACCELERATION SPECTRUM

POINT	PERIOD	VALUE
1	0.000	0.3260
2	0.086	0.8140
3	0.418	0.8140
4	0.500	0.6960
5	0.600	0.5800
6	0.700	0.5000
7	0.800	0.4350
8	0.900	0.3870
9	1.000	0.3480
10	1.100	0.3160
11	1.200	0.2900
12	1.300	0.2670
13	1.400	0.2490
14	1.500	0.2320
15	2.000	0.1740
16	2.500	0.1390
17	3.000	0.1160
18	3.500	0.0990
19	4.000	0.0870
20	4.500	0.0770
21	5.000	0.0700

```
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Shelby Co. Holmes St. over CSX RR
_________________________________________________________________

                 LOADING DATA INPUT (CONTINUED)

-----------------------------------------------------------------

        2.0+
           !                   USER INPUT ACCELERATION SPECTRUM
        1.9+
           !
        1.8+
           !
        1.7+
           !
        1.6+
   A       !
   C    1.5+
   C       !
   E    1.4+
   L       !
   E    1.3+
   R       !
   A    1.2+
   T       !
   I    1.1+
   O       !
   N    1.0+
           !
        0.9+
   I       !
        0.8+ X  X
           !
   X    0.7+     X
   G       !
        0.6+      X
           !
        0.5+       X
           !        X
        0.4+         X
           !X         X
        0.3+           XX
           !             XXX
        0.2+
           !                    X    X
        0.1+                              X    X    X    X
           !                                                  X
        0.0+
            +---------+---------+---------+---------+---------+
            0         1         2         3         4         5
                           PERIOD  (SECONDS)
```

LOADINGS DATA INPUT (CONTINUED)

LOAD CASE AND LOAD CASE COMBINATION INFORMATION

LOAD CASE/COMB	........DIRECTION FACTORS....... X	Y	Z	DESCRIPTION
1	0.067	0.000	-0.998	Longitudinal
2	0.998	0.000	0.067	Transverse
3				1.0*Long + 0.3*Trans
4				0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - -   (Version 5.0.5)   07-JUL-06
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Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

VIBRATION CHARACTERISTICS

			PARTICIPATION FACTORS			% OF TOTAL MASS		
MODE	PERIOD	CS	Long	Vert	Tran	Long	Vert	Tran
1	0.362	0.81	1.788	-0.022	8.476	4.157	0.001	93.426
2	0.266	0.81	-0.010	-0.004	0.699	4.157	0.001	94.061
3	0.250	0.81	8.440	-0.119	-1.768	96.803	0.019	98.127
4	0.180	0.81	0.158	2.520	-0.021	96.836	8.278	98.128
5	0.129	0.81	0.039	-5.353	-0.020	96.838	45.540	98.128
6	0.101	0.81	0.135	4.709	-0.015	96.861	74.377	98.128
7	0.070	0.72	-0.043	-0.546	0.004	96.864	74.764	98.128
8	0.049	0.60	-0.009	-1.004	0.007	96.864	76.076	98.129
9	0.041	0.56	0.006	0.220	0.103	96.864	76.139	98.142

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Imbsen and Associates, Inc.

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

ABUTMENT CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.040	0.016	0.040	0.016	0.000	0.000
	2	0.019	0.072	0.019	0.072	0.000	0.000
	3	0.045	0.038	0.045	0.038	0.000	0.000
	4	0.031	0.077	0.031	0.077	0.000	0.000
ABU 4	1	0.040	0.021	0.040	0.021	0.000	0.000
	2	0.019	0.095	0.019	0.095	0.000	0.000
	3	0.045	0.050	0.045	0.050	0.000	0.000
	4	0.031	0.101	0.031	0.101	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

BENT CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	....LEFT FACE.... TRNSVRSE	....RGHT FACE.... LNGTUDNL	....RGHT FACE.... TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	...OPNNG/CLSNG... TRNSVRSE
BNT 2	1	0.041	0.018	0.041	0.018	0.000	0.000
	2	0.019	0.079	0.019	0.079	0.000	0.000
	3	0.046	0.041	0.046	0.041	0.000	0.000
	4	0.031	0.084	0.031	0.084	0.000	0.000
BNT 3	1	0.041	0.020	0.041	0.020	0.000	0.000
	2	0.019	0.088	0.019	0.088	0.000	0.000
	3	0.047	0.046	0.047	0.046	0.000	0.000
	4	0.031	0.094	0.031	0.094	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - - (Version 5.0.5) 07-JUL-06
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Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

COLUMN CQC FORCES

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	16.8	430.	31.7	395.	31.9	4.2
		2	13.1	192.	150.5	1877.	150.1	16.0
		3	20.8	487.	76.8	958.	76.9	9.0
		4	18.2	321.	160.0	1995.	159.6	17.3
1	TOP	1	13.0	69.	30.4	356.	31.8	4.2
		2	12.2	127.	144.4	1688.	150.0	16.0
		3	16.7	107.	73.7	862.	76.8	9.0
		4	16.1	148.	153.5	1795.	159.5	17.3
2	BOT	1	18.0	444.	34.7	419.	20.6	4.2
		2	6.4	123.	165.0	1993.	66.8	16.0
		3	19.9	481.	84.2	1017.	40.7	9.0
		4	11.8	256.	175.4	2119.	73.0	17.3
2	TOP	1	14.1	50.	33.3	403.	20.6	4.2
		2	5.6	41.	158.7	1919.	66.8	16.0
		3	15.8	62.	80.9	979.	40.6	9.0
		4	9.8	56.	168.7	2039.	73.0	17.3
3	BOT	1	19.3	460.	34.8	420.	29.1	4.2
		2	4.5	94.	165.0	1993.	67.8	16.0
		3	20.7	488.	84.3	1018.	49.4	9.0
		4	10.3	232.	175.4	2119.	76.5	17.3
3	TOP	1	15.3	36.	33.5	405.	29.1	4.2
		2	3.9	47.	158.7	1919.	67.8	16.0
		3	16.4	50.	81.1	981.	49.4	9.0
		4	8.5	58.	168.7	2041.	76.5	17.3
4	BOT	1	20.7	476.	31.7	395.	31.4	4.2
		2	10.4	135.	150.5	1877.	150.0	16.0
		3	23.8	517.	76.8	958.	76.4	9.0
		4	16.6	278.	160.0	1995.	159.4	17.3

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - - (Version 5.0.5) 07-JUL-06
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Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

COLUMN CQC FORCES (CONTINUED)

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 2 (CONTINUED)								
COLUMN 4 (CONTINUED)								
4	TOP	1	16.6	32.	30.4	355.	31.4	4.2
		2	10.0	133.	144.4	1688.	149.9	16.0
		3	19.6	72.	73.7	862.	76.4	9.0
		4	14.9	143.	153.5	1795.	159.3	17.3
BNT 3								
1	BOT	1	17.7	435.	32.4	416.	33.2	2.4
		2	10.1	154.	153.6	1971.	157.6	13.2
		3	20.7	481.	78.5	1008.	80.5	6.4
		4	15.4	284.	163.3	2096.	167.5	13.9
1	TOP	1	13.5	43.	30.9	374.	33.2	2.4
		2	9.2	102.	146.6	1773.	157.5	13.2
		3	16.3	74.	74.9	906.	80.5	6.4
		4	13.2	115.	155.9	1886.	167.5	13.9
2	BOT	1	17.6	434.	35.4	441.	17.5	2.4
		2	5.1	103.	167.9	2090.	60.6	13.2
		3	19.2	465.	85.8	1068.	35.6	6.4
		4	10.4	234.	178.5	2222.	65.8	13.9
2	TOP	1	13.4	40.	33.9	424.	17.4	2.4
		2	4.3	33.	160.7	2008.	60.6	13.2
		3	14.7	50.	82.1	1027.	35.6	6.4
		4	8.3	45.	170.9	2136.	65.8	13.9
3	BOT	1	17.6	434.	35.4	441.	14.3	2.4
		2	4.0	88.	167.9	2090.	61.5	13.2
		3	18.8	461.	85.8	1068.	32.7	6.4
		4	9.3	218.	178.6	2223.	65.7	13.9

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - - (Version 5.0.5) 07-JUL-06
Imbsen and Associates, Inc.

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

COLUMN CQC FORCES (CONTINUED)

CL LOC	LC	LNGITUDNL SHEAR	LNGITUDNL MOMENT	TRANSVRSE SHEAR	TRANSVRSE MOMENT	AXIAL	TORSION
BNT 3 (CONTINUED)							
COLUMN 3 (CONTINUED)							
3 TOP	1	13.4	40.	33.9	424.	14.3	2.4
	2	3.4	37.	160.7	2009.	61.4	13.2
	3	14.4	51.	82.1	1027.	32.7	6.4
	4	7.5	49.	170.9	2136.	65.7	13.9
4 BOT	1	17.7	435.	32.4	416.	33.3	2.4
	2	8.5	121.	153.6	1971.	157.5	13.2
	3	20.2	471.	78.5	1008.	80.5	6.4
	4	13.8	252.	163.3	2096.	167.5	13.9
4 TOP	1	13.5	44.	30.9	374.	33.3	2.4
	2	8.0	106.	146.6	1773.	157.4	13.2
	3	15.9	75.	74.9	906.	80.5	6.4
	4	12.1	119.	155.9	1886.	167.4	13.9

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

ABUTMENT CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
ABU 1	1	8.7	860.6	180.6	877.3	60.5
	2	2.5	218.8	278.0	209.3	285.3
	3	9.5	926.3	264.0	940.1	146.0
	4	5.1	477.0	332.2	472.5	303.4
ABU 4	1	14.0	867.3	197.4	886.0	78.1
	2	2.8	166.3	374.4	178.1	369.0
	3	14.9	917.1	309.8	939.4	188.8
	4	7.0	426.5	433.7	443.9	392.4

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

ABUTMENT FOUNDATION SPRING CQC FORCES

ABUT	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
1	1	877.3	0.0	60.5	0.0	0.0	0.0
	2	209.3	0.0	285.3	0.0	0.0	0.0
	3	940.1	0.0	146.0	0.0	0.0	0.0
	4	472.5	0.0	303.4	0.0	0.0	0.0
4	1	886.0	0.0	78.1	0.0	0.0	0.0
	2	178.1	0.0	369.0	0.0	0.0	0.0
	3	939.4	0.0	188.8	0.0	0.0	0.0
	4	443.9	0.0	392.4	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

BENT CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
BNT 2	1	35.5	107.8	97.0	110.1	94.4
	2	7.4	98.2	438.0	24.1	448.3
	3	37.7	137.3	228.4	117.3	228.9
	4	18.1	130.6	467.1	57.1	476.6
BNT 3	1	13.8	115.7	95.1	117.6	92.7
	2	2.8	96.5	429.3	24.0	439.4
	3	14.7	144.6	223.9	124.8	224.6
	4	7.0	131.2	457.9	59.3	467.2

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

BENT BEARING CQC FORCES

BRNG	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
BNT 2							
1	1	14.4	4.1	12.0	0.0	0.0	0.0
	2	17.1	9.1	53.7	0.0	0.0	0.0
	3	19.6	6.8	28.1	0.0	0.0	0.0
	4	21.4	10.3	57.3	0.0	0.0	0.0
2	1	14.2	4.1	12.0	0.0	0.0	0.0
	2	15.7	6.5	54.0	0.0	0.0	0.0
	3	18.8	6.0	28.2	0.0	0.0	0.0
	4	19.9	7.7	57.6	0.0	0.0	0.0
3	1	13.9	4.2	12.1	0.0	0.0	0.0
	2	14.3	3.9	54.3	0.0	0.0	0.0
	3	18.2	5.3	28.3	0.0	0.0	0.0
	4	18.4	5.2	57.9	0.0	0.0	0.0
4	1	13.6	4.3	12.1	0.0	0.0	0.0
	2	12.9	1.5	54.6	0.0	0.0	0.0
	3	17.5	4.8	28.5	0.0	0.0	0.0
	4	17.0	2.8	58.2	0.0	0.0	0.0
5	1	13.3	4.6	12.2	0.0	0.0	0.0
	2	11.6	1.7	54.9	0.0	0.0	0.0
	3	16.8	5.1	28.6	0.0	0.0	0.0
	4	15.6	3.1	58.6	0.0	0.0	0.0
6	1	13.1	4.9	12.2	0.0	0.0	0.0
	2	10.4	4.1	55.2	0.0	0.0	0.0
	3	16.2	6.1	28.8	0.0	0.0	0.0
	4	14.4	5.6	58.9	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

- - - - WinSEISAB - - - - (Version 5.0.5) 07-JUL-06
Imbsen and Associates, Inc.

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

BENT BEARING CQC FORCES (CONTINUED)

BRNG	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
BNT 2 (CONTINUED)							
7	1	12.8	5.2	12.3	0.0	0.0	0.0
	2	9.3	6.7	55.5	0.0	0.0	0.0
	3	15.6	7.2	28.9	0.0	0.0	0.0
	4	13.2	8.3	59.2	0.0	0.0	0.0
8	1	12.6	5.6	12.3	0.0	0.0	0.0
	2	8.4	9.3	55.8	0.0	0.0	0.0
	3	15.1	8.4	29.1	0.0	0.0	0.0
	4	12.2	11.0	59.5	0.0	0.0	0.0
BNT 3							
1	1	14.6	3.0	11.7	0.0	0.0	0.0
	2	15.5	8.4	52.9	0.0	0.0	0.0
	3	19.2	5.5	27.6	0.0	0.0	0.0
	4	19.8	9.3	56.4	0.0	0.0	0.0
2	1	14.5	2.5	11.8	0.0	0.0	0.0
	2	14.5	6.0	53.1	0.0	0.0	0.0
	3	18.9	4.3	27.7	0.0	0.0	0.0
	4	18.8	6.7	56.6	0.0	0.0	0.0
3	1	14.5	2.1	11.8	0.0	0.0	0.0
	2	13.5	3.6	53.3	0.0	0.0	0.0
	3	18.6	3.2	27.8	0.0	0.0	0.0
	4	17.8	4.2	56.9	0.0	0.0	0.0
4	1	14.5	1.8	11.9	0.0	0.0	0.0
	2	12.5	1.2	53.6	0.0	0.0	0.0
	3	18.2	2.2	27.9	0.0	0.0	0.0
	4	16.9	1.7	57.1	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

BENT BEARING CQC FORCES (CONTINUED)

BRNG	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
BNT 3 (CONTINUED)							
5	1	14.4	1.7	11.9	0.0	0.0	0.0
	2	11.6	1.4	53.8	0.0	0.0	0.0
	3	17.9	2.1	28.0	0.0	0.0	0.0
	4	15.9	1.9	57.4	0.0	0.0	0.0
6	1	14.4	1.7	12.0	0.0	0.0	0.0
	2	10.8	3.7	54.0	0.0	0.0	0.0
	3	17.6	2.8	28.2	0.0	0.0	0.0
	4	15.1	4.3	57.6	0.0	0.0	0.0
7	1	14.4	1.9	12.0	0.0	0.0	0.0
	2	10.0	6.2	54.2	0.0	0.0	0.0
	3	17.4	3.8	28.3	0.0	0.0	0.0
	4	14.3	6.7	57.8	0.0	0.0	0.0
8	1	14.4	2.2	12.1	0.0	0.0	0.0
	2	9.2	8.6	54.4	0.0	0.0	0.0
	3	17.1	4.8	28.4	0.0	0.0	0.0
	4	13.6	9.3	58.1	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

```
                      W     W  III  N   N
                      W     W   I   NN  N
                      W  W  W   I   N N N
                       W W W    I   N  NN
                        W W    III  N   N

 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS    AAAAAAA   BBBBBBBB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AAAAAAAAA  BBBBBBBBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSS        EE            III     SSS        AA     AA  BB    BBB
 SSSSSS    EEEEE         III      SSSSSS    AAAAAAAAA  BBBBBBBB
  SSSSSS   EEEEE         III       SSSSSS   AAAAAAAAA  BBBBBBBB
      SSS  EE            III           SSS  AA     AA  BB    BBB
SS     SS  EE            III     SS     SS  AA     AA  BB     BB
SSSSSSSSS  EEEEEEEEE  IIIIIIIII  SSSSSSSSS  AA     AA  BBBBBBBBB
 SSSSSSS   EEEEEEEEE  IIIIIIIII   SSSSSSS   AA     AA  BBBBBBBB


****************************************************************
*                                                              *
*                          WinSeisab                           *
*                                                              *
*                 Seismic Analysis of Bridges                  *
*                                                              *
*       Version 5.0.5                    Release 07/2002       *
*                                                              *
*                                                              *
*                                                              *
*                   Imbsen Software Systems                    *
*                                                              *
*                        www.Imbsen.com                        *
*                                                              *
*                                                              *
*                                                              *
*           Windows (GUI) By:  CV-McBridge Software            *
*                                                              *
*                     www.CV-McBridge.com                      *
*                                                              *
*                                                              *
*                                                              *
*------------------------- Licensed To: -------------------------*
*                                                              *
*           Tennessee Department of Transportation             *
*                                                              *
*------------------------- Mar 27, 2003 ------------------------*
*                                                              *
*                                                              *
*                                                              *
*                 Written By:  Roy Imbsen                      *
*                              Jon Lea                         *
*                              Clark Verkler                   *
*                              James Gates                     *
*                                                              *
****************************************************************

Date:  20-JUL-06                                Time:  14:22:30
```

Echo of this file:
P:\STRUC_DS\Region IV\IYH\PaulChambers\Shelby Co. Holmes St\Holmes 2.ssb

---

```
# WinSeisab Version 5.0.5

# Length_Unit ft
# Force_Unit  kip
# Time_Unit   sec

# Length_Prec     4
# Force_Prec      4
# Time_Prec       4
# Area_Prec       6
# Volume_Prec     4
# Inertia_Prec    4
# Moment_Prec     4
# LinWeight_Prec  4
# Stress_Prec     4
# Density_Prec    4
# Trans_K_Prec    4
# Rotat_K_Prec    4
# Couple_K_Prec   8
# Accel_Prec      2

# ShowExponentialK   No


SEISAB "Shelby Co. Holmes St. over CSX RR"
   RESPONSE SPECTRUM ANALYSIS
   SUPERSTRUCTURE JOINTS   3
   COLUMN JOINTS   2
   OUTPUT LEVEL   0
   BLOCKING FACTOR   1



ALIGNMENT
   STATION   0.0
   COORDINATES   N  0.0   E  0.0
   BEARING   N 90 00 00 E



SPANS
   LENGTHS         40.0  60.0  48.0
   AREA            65.875  65.875  65.875
   I11             9.604  9.604  9.604
   I22             24249.1  24249.1  24249.1
   I33             86.7021  86.7021  86.7021
   A22             0.0  0.0  0.0
   A33             0.0  0.0  0.0
   DENSITY         0.15  0.15  0.15
   WEIGHT          3.078  3.078  3.078
   E               478160.78  478160.78  478160.78
   PRATIO          0.2  0.2  0.2
```

```
DESCRIBE
   COLUMN   'Type 1'
      AREA            7.069
      I11             3.976
      I22             1.988
      I33             1.988
      A22             0.0
      A33             0.0
      DENSITY         0.15
      E               617303.0
      PRATIO          0.2



   SPECIAL CAP   'Type 1'
      I33       21.333
      A22       16.0
      E         478161.0
      PRATIO    0.2



   BEARING ELEMENT   'elasto'
      KF1F1   1000000000.0
      KF2F2   1000000000.0
      KF3F3   1000000000.0
      KM1M1   0.0
      KM2M2   0.0
      KM3M3   0.0



ABUTMENT STATION 0.0
   BEARING  N 12 3 36 W  N 12 3 36 W

   ELEVATION TOP 312.082 AT ABUTMENT 1
   ELEVATION TOP 313.812 AT ABUTMENT 4

   CONNECTION PIN    AT ABUTMENT 1
   CONNECTION PIN    AT ABUTMENT 4



BENT
   BEARING  N 12 3 36 W  N 12 3 36 W

   ELEVATION TOP       313.042  313.802
   ELEVATION BEARINGS  310.222  310.982
   ELEVATION CAP       308.097  308.857
   ELEVATION BOTTOM    280.5  280.5
   WEIGHT  192.4125  192.4125

   COLUMN  SKEWED LAYOUT  'Type 1'  18.333  'Type 1'  18.333  'Type 1' -
           18.333  'Type 1'  AT BENT 2
   COLUMN  SKEWED LAYOUT  'Type 1'  18.333  'Type 1'  18.333  'Type 1' -
           18.333  'Type 1'  AT BENT 3

   COLUMN TOP    FIX    AT BENT 2
   COLUMN BOTTOM FIX    AT BENT 2
```

```
   COLUMN TOP    FIX    AT BENT 3
   COLUMN BOTTOM FIX    AT BENT 3

   SPECIAL CAP 'Type 1'  AT BENT 2
   SPECIAL CAP 'Type 1'  AT BENT 3

   CONNECTION  BEARING ELEMENT NORMAL LAYOUT  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               AT BENT 2
   CONNECTION  BEARING ELEMENT NORMAL LAYOUT  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               8.6927  'elasto'  8.6927  'elasto'  8.6927  'elasto' -
               AT BENT 3


FOUNDATION
   AT ABUTMENT  1 4
      SPRING CONSTANTS
         KF1F1   22119.5
C --     KF2F2   fixed
         KF3F3   3840.0
C --     KM1M1   fixed
C --     KM2M2   fixed
C --     KM3M3   fixed
      WEIGHT  0.0



LOADS
   USE FIXED NOTATION FOR VIBRATION INFO
   USE FIXED NOTATION FOR DISPLACEMENTS
   USE FIXED NOTATION FOR FORCES

   RESPONSE SPECTRUM
      COMBINATION FACTOR   0.3
C --  MODE SHAPES  -- using default
      DAMPING COEFFICIENT   0.05

      AASHTO CURVE
         SOIL TYPE II
         ACCELERATION COEFFICIENT   0.2

      GRAVITY   32.2

FINISH
```

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- - - - WinSEISAB - - - -       (Version 5.0.5)        20-JUL-06
                                    Imbsen and Associates, Inc.

Shelby Co. Holmes St. over CSX RR
________________________________________________________________

                      WinSEISAB DATA INPUT

----------------------------------------------------------------


      ****************************************************
      *                                                  *
      *                                                  *
      *                                                  *
      *                                                  *
      *                RESPONSE SPECTRUM                 *
      *                                                  *
      *                                                  *
      *                                                  *
      *                                                  *
      ****************************************************


           THE BRIDGE MODEL GENERATED IS BASED ON:

                3 INTERMEDIATE JOINT(S) ON EACH SPAN

                2 INTERMEDIATE JOINT(S) ON EACH COLUMN
```

Shelby Co. Holmes St. over CSX RR

ALIGNMENT DATA INPUT

INITIAL REFERENCE POINT AND ALIGNMENT

STATION	STATION COORDINATE		....OFFSET.... DIR	VALUE	BEARING
0.00	N	0.00		0.00	N 90 0 0 E
	E	0.00			

Shelby Co. Holmes St. over CSX RR

SPAN DATA INPUT

SEGMENT LENGTHS / SECTION AND MATERIAL PROPERTIES

SEGMENT	LENGTH	AREA	I11	I22	I33
SPAN 1	40.00				
1	40.00	65.87	9.60	24249.10	86.70
SPAN 2	60.00				
1	60.00	65.87	9.60	24249.10	86.70
SPAN 3	48.00				
1	48.00	65.87	9.60	24249.10	86.70

SEGMENT	A22	A33	ELASTIC MODULUS	P-RATIO	DENSITY	WT/LENGTH
SPAN 1						
1	0.0	0.0	478161.	0.20	0.150	3.078
SPAN 2						
1	0.0	0.0	478161.	0.20	0.150	3.078
SPAN 3						
1	0.0	0.0	478161.	0.20	0.150	3.078

DESCRIBE DATA INPUT

COLUMN INFORMATION

NO. 1 TITLE: Type 1
INFORMATION:

SEGMENT	LENGTH	AREA	I11	I22	I33
1	0.00	7.07	3.98	1.99	1.99

SEGMENT	A22	A33	ELASTIC MODULUS	P-RATIO	DENSITY
1	0.0	0.0	617303.	0.20	0.15

SPECIAL BENT CAP INFORMATION

NO. 1 TITLE: Type 1
INFORMATION:
DENSITY:
ELASTIC MODULUS: 478161.
POISSONS RATIO: 0.20

	AREA	I11	I22	I33	A22	A33
SPECIAL CAP MEMBER				21.33	16.0	

DESCRIBE DATA INPUT (CONTINUED)

BEARING ELEMENT INFORMATION

NO. 1 TITLE: elasto
INFORMATION:

KF1F1	KF1F2	KF1F3	KF1M1	KF1M2	KF1M3
1.0E+09	0.	0.	0.	0.	0.

KF2F2	KF2F3	KF2M1	KF2M2	KF2M3
1.0E+09	0.	0.	0.	0.

KF3F3	KF3M1	KF3M2	KF3M3
1.0E+09	0.	0.	0.

KM1M1	KM1M2	KM1M3
0.	0.	0.

KM2M2	KM2M3
0.	0.

KM3M3
0.

Shelby Co. Holmes St. over CSX RR

ABUTMENT DATA INPUT

GEOMETRY AND GENERAL INFORMATION

ABT	STATION	BEARING	SUPER CG ELEVATION	WALL BOT ELEVATION	ABUTMENT CONNECTION
1	0.00	N 12 3 36 W	312.08		PIN
4		N 12 3 36 W	313.81		PIN

Shelby Co. Holmes St. over CSX RR

BENT DATA INPUT

BENT GEOMETRY, WEIGHT AND CAP INFORMATION

BENT	BEARING	WEIGHT	CAP TITLE	NOTE
2	N 12 3 36 W	192.41	Type 1	
3	N 12 3 36 W	192.41	Type 1	

BENT ELEVATION INFORMATION

BENT	SUPER CG ELEVATION	DIAPHRAGM ELEVATION	BEAR. ELMT ELEVATION	BENT CAP ELEVATION	FOOTING ELEVATION
2	313.04		310.22	308.10	280.50
3	313.80		310.98	308.86	280.50

BENT CONNECTIVITY INFORMATION

BENT	SUPERSTR CONTINUITY	BENT TO SUPERSTR CONNECTION TYPE
2	CONTINUOUS	BEARING
3	CONTINUOUS	BEARING

BENT DATA INPUT (CONTINUED)

BENT BEARING ELEMENT LAYOUT

BENT	LOCATION	SPACING	BE	TITLE	DISTANCE
2		NORMAL	1	elasto	8.69
			2	elasto	8.69
			3	elasto	8.69
			4	elasto	8.69
			5	elasto	8.69
			6	elasto	8.69
			7	elasto	8.69
			8	elasto	
3		NORMAL	1	elasto	8.69
			2	elasto	8.69
			3	elasto	8.69
			4	elasto	8.69
			5	elasto	8.69
			6	elasto	8.69
			7	elasto	8.69
			8	elasto	

BENT COLUMN LAYOUT

					....GROUP	OFFSET...	
BENT	SPACING	COL	TITLE	DISTANCE	SPCNG	DIR	OFFSET
2	SKEWED	1	Type 1	18.33			0.00
		2	Type 1	18.33			
		3	Type 1	18.33			
		4	Type 1				
3	SKEWED	1	Type 1	18.33			0.00
		2	Type 1	18.33			
		3	Type 1	18.33			
		4	Type 1				

Shelby Co. Holmes St. over CSX RR

BENT DATA INPUT (CONTINUED)

BENT COLUMN END INFORMATION

		TOP			BOTTOM		
BNT	COL	LT LT ASSTMM	BRG ELMT	JNT SIZE	LT LT ASSTMM	BRG ELMT	JNT SIZE
2	1			0.00			0.00
	2			0.00			0.00
	3			0.00			0.00
	4			0.00			0.00
3	1			0.00			0.00
	2			0.00			0.00
	3			0.00			0.00
	4			0.00			0.00

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Shelby Co. Holmes St. over CSX RR

FOUNDATION DATA

ABUTMENT FOUNDATION INFORMATION

ABUTMENT	WEIGHT	ROT. ANGLE	FOUNDATION TYPE	FTNG TITLE
1	0.00	0.00	SPRING CONSTANTS	
4	0.00	0.00	SPRING CONSTANTS	

ABUTMENT FOUNDATION SPRING CONSTANTS

ABUT	KF1F1	KF1F2	KF1F3	KF1M1	KF1M2	KF1M3
1	22120.		0.			

KF2F2	KF2F3	KF2M1	KF2M2	KF2M3
DOF FIXD				

KF3F3	KF3M1	KF3M2	KF3M3
3840.			

KM1M1	KM1M2	KM1M3
DOF FIXD		

KM2M2	KM2M3
DOF FIXD	

KM3M3
DOF FIXD

FOUNDATION DATA (CONTINUED)

ABUTMENT FOUNDATION SPRING CONSTANTS (CONTINUED)

ABUT	KF1F1	KF1F2	KF1F3	KF1M1	KF1M2	KF1M3
4	22120.		0.			

KF2F2	KF2F3	KF2M1	KF2M2	KF2M3
DOF FIXD				

KF3F3	KF3M1	KF3M2	KF3M3
3840.			

KM1M1	KM1M2	KM1M3
DOF FIXD		

KM2M2	KM2M3
DOF FIXD	

KM3M3
DOF FIXD

LOADINGS DATA INPUT

THE NUMBER OF MODE SHAPES FOUND WILL BE 9

ACCELERATION SPECTRUM INFORMATION

SPECTRUM TYPE = AASHTO
SOIL TYPE = II
ACCELERATION COEFFICIENT = 0.20
ACCELERATION DUE TO GRAVITY = 32.20

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Shelby Co. Holmes St. over CSX RR
________________________________________________________________

                 LOADING DATA INPUT (CONTINUED)

----------------------------------------------------------------

        2.0+
           !                       AASHTO ACCELERATION SPECTRUM
        1.9+
           !
        1.8+
           !
        1.7+
           !
        1.6+
   A       !
   C    1.5+
   C       !
   E    1.4+
   L       !
   E    1.3+
   R       !
   A    1.2+
   T       !
   I    1.1+
   O       !
   N    1.0+
           !
        0.9+
   I       !
        0.8+
           !
   X    0.7+
   G       !
        0.6+
           !
        0.5+XXXXX
           !     X
        0.4+      X
           !       XX
        0.3+         XX
           !           XXXX
        0.2+               XXXXXXX
           !                      XXXXXXXXXXXXX
        0.1+                                   XXXXXXXXXXXXXXXX
           !
        0.0+
            +---------+---------+---------+---------+---------+
            0         1         2         3         4         5
                           PERIOD  (SECONDS)
```

Shelby Co. Holmes St. over CSX RR

LOADINGS DATA INPUT (CONTINUED)

LOAD CASE AND LOAD CASE COMBINATION INFORMATION

LOAD CASE/COMB	........DIRECTION FACTORS....... X	Y	Z	DESCRIPTION
1	1.000	0.000	0.000	Longitudinal
2	0.000	0.000	1.000	Transverse
3				1.0*Long + 0.3*Trans
4				0.3*Long + 1.0*Trans

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Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

VIBRATION CHARACTERISTICS

MODE	PERIOD	CS	PARTICIPATION FACTORS Long	Vert	Tran	% OF TOTAL MASS Long	Vert	Tran
1	0.479	0.47	1.792	-0.022	8.538	4.137	0.001	93.858
2	0.297	0.50	0.063	0.001	-0.196	4.142	0.001	93.908
3	0.256	0.50	8.474	-0.117	-1.777	96.603	0.018	97.973
4	0.181	0.50	0.156	2.665	-0.023	96.635	9.161	97.974
5	0.131	0.50	0.047	-5.389	-0.022	96.638	46.552	97.974
6	0.103	0.50	0.138	4.634	-0.016	96.662	74.206	97.975
7	0.071	0.50	0.035	-0.586	-0.012	96.664	74.649	97.975
8	0.068	0.50	0.933	-0.007	-0.200	97.786	74.649	98.026
9	0.065	0.50	0.979	0.058	-0.209	99.019	74.653	98.083

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

ABUTMENT CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	TRNSVRSE	....RGHT FACE.... LNGTUDNL	TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	TRNSVRSE
ABU 1	1	0.026	0.017	0.026	0.017	0.000	0.000
	2	0.019	0.081	0.019	0.081	0.000	0.000
	3	0.031	0.042	0.031	0.042	0.000	0.000
	4	0.026	0.086	0.026	0.086	0.000	0.000
ABU 4	1	0.026	0.019	0.026	0.019	0.000	0.000
	2	0.019	0.087	0.019	0.087	0.000	0.000
	3	0.031	0.045	0.031	0.045	0.000	0.000
	4	0.026	0.093	0.026	0.093	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

RESPONSE SPECTRUM RESULTS

BENT CQC DISPLACEMENTS

ITEM	LC	....LEFT FACE.... LNGTUDNL	....LEFT FACE.... TRNSVRSE	....RGHT FACE.... LNGTUDNL	....RGHT FACE.... TRNSVRSE	...OPNNG/CLSNG... LNGTUDNL	...OPNNG/CLSNG... TRNSVRSE
BNT 2	1	0.026	0.018	0.026	0.018	0.000	0.000
	2	0.019	0.084	0.019	0.084	0.000	0.000
	3	0.032	0.044	0.032	0.044	0.000	0.000
	4	0.027	0.090	0.027	0.090	0.000	0.000
BNT 3	1	0.027	0.019	0.027	0.019	0.000	0.000
	2	0.019	0.087	0.019	0.087	0.000	0.000
	3	0.032	0.045	0.032	0.045	0.000	0.000
	4	0.027	0.093	0.027	0.093	0.000	0.000

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

COLUMN CQC FORCES

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 2								
1	BOT	1	6.2	122.	12.3	170.	13.9	0.9
		2	2.0	33.	58.4	810.	65.7	2.6
		3	6.8	132.	29.8	413.	33.6	1.7
		4	3.9	70.	62.1	861.	69.9	2.9
1	TOP	1	2.4	20.	11.3	158.	13.9	0.9
		2	1.4	19.	54.0	752.	65.7	2.6
		3	2.8	26.	27.6	383.	33.6	1.7
		4	2.1	25.	57.4	799.	69.9	2.9
2	BOT	1	6.4	125.	12.8	175.	10.4	0.9
		2	1.5	28.	61.1	834.	10.1	2.6
		3	6.9	133.	31.1	425.	13.4	1.7
		4	3.4	65.	64.9	887.	13.2	2.9
2	TOP	1	2.5	18.	11.9	168.	10.3	0.9
		2	0.7	7.	56.6	799.	10.1	2.6
		3	2.7	20.	28.9	407.	13.4	1.7
		4	1.5	12.	60.2	850.	13.2	2.9
3	BOT	1	6.6	128.	12.8	175.	16.7	0.9
		2	1.5	27.	61.1	834.	12.0	2.6
		3	7.1	136.	31.2	426.	20.4	1.7
		4	3.4	66.	64.9	887.	17.1	2.9
3	TOP	1	2.7	16.	11.9	168.	16.7	0.9
		2	0.7	7.	56.6	800.	12.0	2.6
		3	2.9	18.	28.9	408.	20.3	1.7
		4	1.5	12.	60.2	850.	17.1	2.9
4	BOT	1	6.8	131.	12.3	170.	13.8	0.9
		2	2.0	33.	58.4	810.	65.5	2.6
		3	7.4	141.	29.8	413.	33.5	1.7
		4	4.0	72.	62.1	861.	69.7	2.9

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

RESPONSE SPECTRUM RESULTS

COLUMN CQC FORCES (CONTINUED)

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 2 (CONTINUED)								
COLUMN 4 (CONTINUED)								
4	TOP	1	2.9	15.	11.3	158.	13.8	0.9
		2	1.4	19.	54.0	752.	65.5	2.6
		3	3.3	21.	27.6	383.	33.4	1.7
		4	2.3	23.	57.4	799.	69.7	2.9
BNT 3								
1	BOT	1	6.5	125.	11.7	166.	13.5	0.3
		2	1.5	28.	55.8	793.	64.0	0.9
		3	7.0	133.	28.4	404.	32.7	0.6
		4	3.5	65.	59.3	843.	68.0	1.0
1	TOP	1	2.4	16.	10.7	154.	13.4	0.3
		2	0.7	7.	51.1	733.	64.0	0.9
		3	2.6	18.	26.1	374.	32.6	0.6
		4	1.4	11.	54.3	780.	68.0	1.0
2	BOT	1	6.5	124.	12.2	171.	8.7	0.3
		2	1.4	26.	58.2	816.	8.0	0.9
		3	6.9	132.	29.7	416.	11.1	0.6
		4	3.3	64.	61.9	867.	10.6	1.0
2	TOP	1	2.4	16.	11.2	164.	8.7	0.3
		2	0.5	4.	53.5	779.	8.0	0.9
		3	2.6	17.	27.3	397.	11.0	0.6
		4	1.3	9.	56.9	828.	10.6	1.0
3	BOT	1	6.4	123.	12.2	171.	4.5	0.3
		2	1.3	26.	58.2	816.	8.0	0.9
		3	6.8	131.	29.7	416.	6.9	0.6
		4	3.3	63.	61.9	867.	9.3	1.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

RESPONSE SPECTRUM RESULTS

COLUMN CQC FORCES (CONTINUED)

CL	LOC	LC	....LNGITUDNL.... SHEAR	MOMENT	....TRANSVRSE.... SHEAR	MOMENT	AXIAL	TORSION
BNT 3 (CONTINUED)								
COLUMN 3 (CONTINUED)								
3	TOP	1	2.4	16.	11.2	164.	4.5	0.3
		2	0.5	4.	53.5	779.	8.0	0.9
		3	2.5	18.	27.3	397.	6.9	0.6
		4	1.2	9.	56.9	828.	9.3	1.0
4	BOT	1	6.4	122.	11.7	166.	13.5	0.3
		2	1.4	26.	55.8	793.	64.0	0.9
		3	6.8	130.	28.4	404.	32.7	0.6
		4	3.3	62.	59.3	843.	68.0	1.0
4	TOP	1	2.3	17.	10.7	154.	13.5	0.3
		2	0.6	7.	51.1	733.	64.0	0.9
		3	2.5	19.	26.1	374.	32.6	0.6
		4	1.3	12.	54.3	780.	68.0	1.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

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Imbsen and Associates, Inc.

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

ABUTMENT CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
ABU 1	1	4.9	557.3	130.9	568.6	67.1
	2	1.5	140.1	311.5	121.5	319.2
	3	5.4	599.4	224.4	605.0	162.9
	4	3.0	307.3	350.7	292.1	339.3
ABU 4	1	9.5	559.4	136.8	571.4	71.8
	2	2.0	127.8	338.4	119.4	341.5
	3	10.1	597.8	238.3	607.2	174.3
	4	4.8	295.7	379.4	290.8	363.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

ABUTMENT FOUNDATION SPRING CQC FORCES

ABUT	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
1	1	568.6	0.0	67.1	0.0	0.0	0.0
	2	121.5	0.0	319.2	0.0	0.0	0.0
	3	605.0	0.0	162.9	0.0	0.0	0.0
	4	292.1	0.0	339.3	0.0	0.0	0.0
4	1	571.4	0.0	71.8	0.0	0.0	0.0
	2	119.4	0.0	341.5	0.0	0.0	0.0
	3	607.2	0.0	174.3	0.0	0.0	0.0
	4	290.8	0.0	363.0	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

BENT CQC FORCES

ITEM	LC	VERT SHEAR	W/R TO BRIDGE C.L. LONGITUDNL	W/R TO BRIDGE C.L. TRANSVERSE	W/R TO ITEM C.L. NORMAL	W/R TO ITEM C.L. PARALLEL
BNT 2	1	23.3	94.2	32.5	96.2	26.0
	2	5.1	33.0	121.2	20.3	124.0
	3	24.8	104.1	68.9	102.3	63.2
	4	12.0	61.3	131.0	49.1	131.8
BNT 3	1	9.9	96.4	31.0	98.7	22.9
	2	2.0	31.0	106.3	20.6	108.8
	3	10.5	105.7	62.9	104.8	55.5
	4	5.0	59.9	115.6	50.2	115.7

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

Shelby Co. Holmes St. over CSX RR

RESPONSE SPECTRUM RESULTS

BENT BEARING CQC FORCES

BRNG	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
BNT 2							
1	1	11.8	2.4	4.0	0.0	0.0	0.0
	2	4.6	2.0	15.0	0.0	0.0	0.0
	3	13.1	3.0	8.6	0.0	0.0	0.0
	4	8.1	2.7	16.2	0.0	0.0	0.0
2	1	11.8	2.5	4.1	0.0	0.0	0.0
	2	4.5	1.4	15.1	0.0	0.0	0.0
	3	13.1	3.0	8.6	0.0	0.0	0.0
	4	8.0	2.2	16.3	0.0	0.0	0.0
3	1	11.8	2.7	4.1	0.0	0.0	0.0
	2	4.3	0.9	15.1	0.0	0.0	0.0
	3	13.1	3.0	8.6	0.0	0.0	0.0
	4	7.9	1.7	16.3	0.0	0.0	0.0
4	1	11.8	2.8	4.1	0.0	0.0	0.0
	2	4.2	0.6	15.1	0.0	0.0	0.0
	3	13.0	3.0	8.6	0.0	0.0	0.0
	4	7.7	1.5	16.4	0.0	0.0	0.0
5	1	11.8	3.0	4.1	0.0	0.0	0.0
	2	4.1	0.8	15.2	0.0	0.0	0.0
	3	13.0	3.2	8.6	0.0	0.0	0.0
	4	7.6	1.7	16.4	0.0	0.0	0.0
6	1	11.8	3.2	4.1	0.0	0.0	0.0
	2	3.9	1.3	15.2	0.0	0.0	0.0
	3	13.0	3.5	8.6	0.0	0.0	0.0
	4	7.5	2.2	16.4	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

RESPONSE SPECTRUM RESULTS

BENT BEARING CQC FORCES (CONTINUED)

BRNG	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
BNT 2 (CONTINUED)							
7	1	11.8	3.3	4.1	0.0	0.0	0.0
	2	3.8	1.8	15.2	0.0	0.0	0.0
	3	12.9	3.9	8.7	0.0	0.0	0.0
	4	7.3	2.8	16.5	0.0	0.0	0.0
8	1	11.8	3.5	4.1	0.0	0.0	0.0
	2	3.7	2.4	15.3	0.0	0.0	0.0
	3	12.9	4.2	8.7	0.0	0.0	0.0
	4	7.2	3.5	16.5	0.0	0.0	0.0
BNT 3							
1	1	12.1	1.7	3.9	0.0	0.0	0.0
	2	4.0	1.8	13.3	0.0	0.0	0.0
	3	13.2	2.3	7.9	0.0	0.0	0.0
	4	7.6	2.3	14.4	0.0	0.0	0.0
2	1	12.1	1.6	3.9	0.0	0.0	0.0
	2	3.9	1.3	13.3	0.0	0.0	0.0
	3	13.2	2.0	7.9	0.0	0.0	0.0
	4	7.6	1.8	14.4	0.0	0.0	0.0
3	1	12.1	1.4	3.9	0.0	0.0	0.0
	2	3.9	0.8	13.3	0.0	0.0	0.0
	3	13.2	1.7	7.9	0.0	0.0	0.0
	4	7.5	1.2	14.4	0.0	0.0	0.0
4	1	12.1	1.3	3.9	0.0	0.0	0.0
	2	3.9	0.4	13.3	0.0	0.0	0.0
	3	13.2	1.4	7.9	0.0	0.0	0.0
	4	7.5	0.7	14.4	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans

RESPONSE SPECTRUM RESULTS

BENT BEARING CQC FORCES (CONTINUED)

BRNG	LC	KF1F1	KF2F2	KF3F3	KM1M1	KM2M2	KM3M3
BNT 3	(CONTINUED)						
5	1	12.1	1.2	3.9	0.0	0.0	0.0
	2	3.9	0.4	13.3	0.0	0.0	0.0
	3	13.2	1.3	7.9	0.0	0.0	0.0
	4	7.5	0.7	14.5	0.0	0.0	0.0
6	1	12.1	1.1	3.9	0.0	0.0	0.0
	2	3.8	0.8	13.3	0.0	0.0	0.0
	3	13.2	1.3	7.9	0.0	0.0	0.0
	4	7.5	1.1	14.5	0.0	0.0	0.0
7	1	12.0	1.0	3.9	0.0	0.0	0.0
	2	3.8	1.3	13.3	0.0	0.0	0.0
	3	13.2	1.4	7.9	0.0	0.0	0.0
	4	7.4	1.6	14.5	0.0	0.0	0.0
8	1	12.0	1.0	3.9	0.0	0.0	0.0
	2	3.8	1.8	13.3	0.0	0.0	0.0
	3	13.2	1.5	7.9	0.0	0.0	0.0
	4	7.4	2.1	14.5	0.0	0.0	0.0

*** LOAD CASE/COMB	DESCRIPTION
1	Longitudinal
2	Transverse
3	1.0*Long + 0.3*Trans
4	0.3*Long + 1.0*Trans