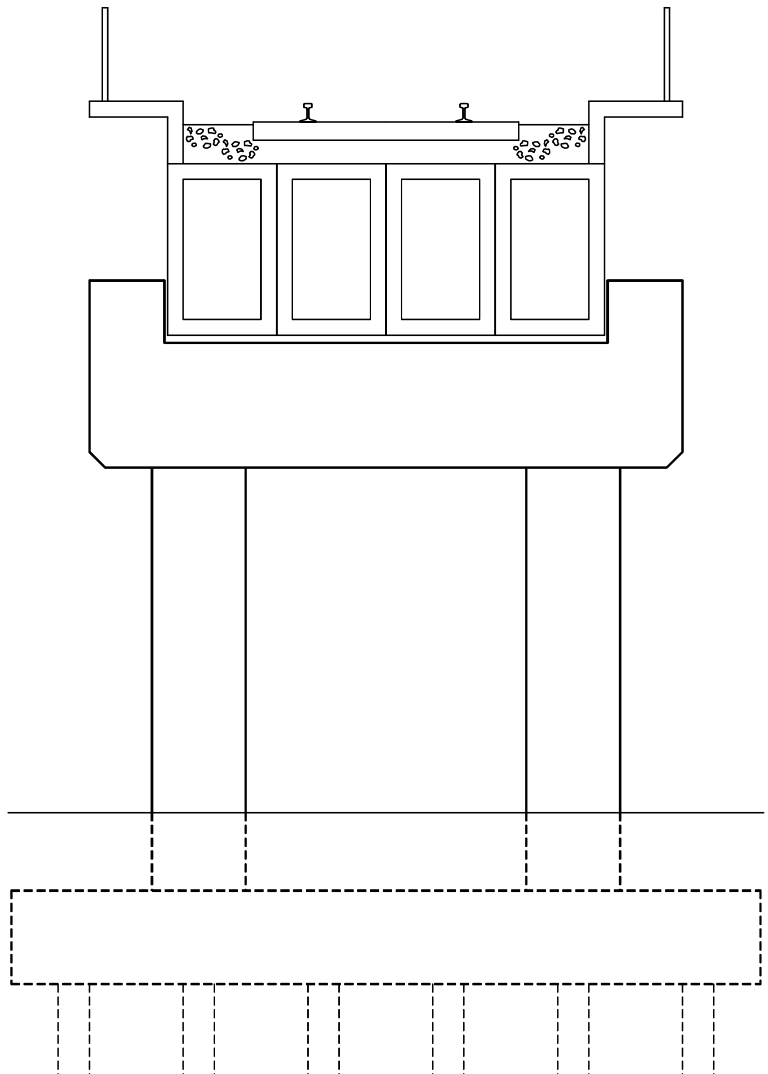

SEISMIC DESIGN EXAMPLE FOR RAILROAD UNDERPASS

Prepared by:

Robert Matthews

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DMJM HARRIS

CONCEPTUAL DESIGN

Design a grade separation underpass structure for a single track over a city street.

Design input:

- Rail

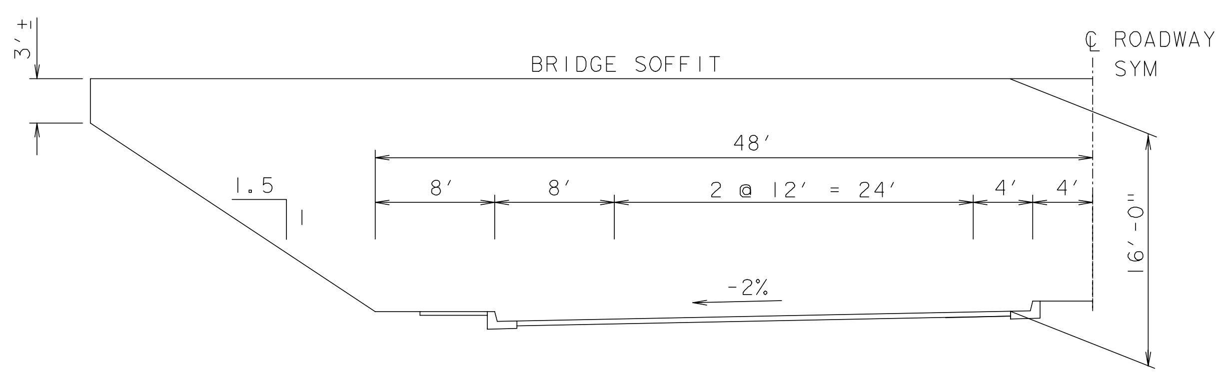
  - Track consists of continuous welded rail (CWR) on timber ties.
  - Track is aligned perpendicular to the street.
  - No maintenance access road is needed at this location.
  - Track is a branchline that has 12 million gross tons of traffic a year.
  - There is no detour around the bridge.
  - Approximately 25% of the traffic is hazardous material.
  - There is not any passenger service on the line.

- Road

  - The roadway is not a community life line.
  - Bridge owner requires a minimum vertical clearance of 16'-0" for roadway.

- Site

  - Location is high seismic area near Los Angeles, California.
  - Soil is silty sand with a friction angle of 32.5 degrees.
  - Bedrock depth exceeds 200 feet.
  - Water table is greater than 100 feet below ground.
  - Pile foundations are required at this location.



Typical Roadway Section at Bridge

Configure bridge:

Bridge length ≅ 2 x (48 + (16 + 0.02 x 36 -.67 - 3) x 1.5 + 1.25) = 138' use 140'
Use (2) 70 foot long spans with central bent.

- Bridge configuration: (AREMA 9-1.4.3.1)

PREFERRED CONFIGURATION	SPECIAL CONSIDERATION
✓Straight bridge alignment	Curved bridge alignment
✓Normal piers	Skewed piers
✓Uniform pier stiffness	Varying pier stiffness
✓Uniform span stiffness	Varying span stiffness
✓Uniform span mass	Varying span mass

- The preferred bridge configuration will be used in all cases

- Superstructure configuration: (AREMA 9-1.4.3.2)

PREFERRED SUPERSTRUCTURE	SPECIAL CONSIDERATION
✓Simple spans	Continuous spans
✓Short spans	Long spans
Light spans	✓Heavy spans
✓No hinges	Intermediate hinges

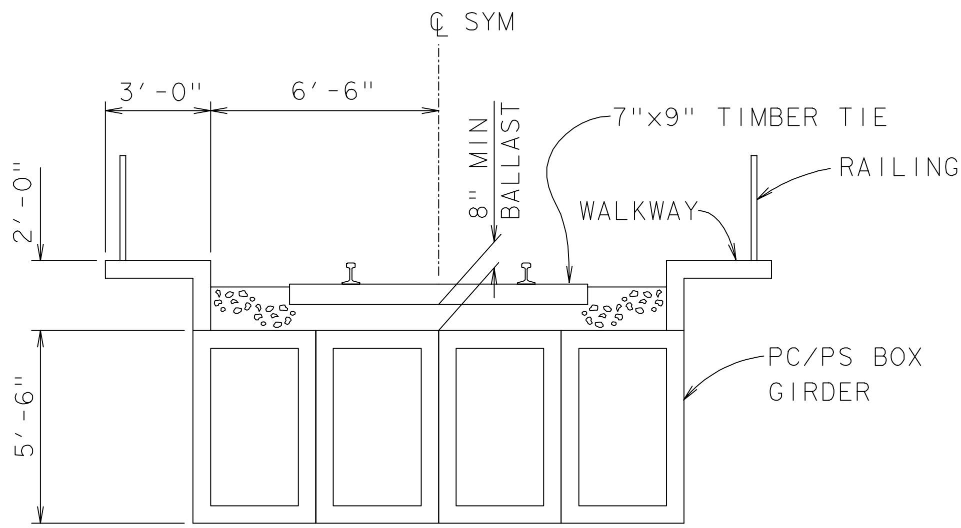
- (2) 70 foot long simple spans will be used.
- Shortest span arrangement is used for given roadway constraints.
- No hinges will be used

- Use precast concrete box girders, even though they are a heavier span. Steel girders are not chosen since they cost about twice that of precast concrete box girders, and the savings in foundation cost is not expected to offset this increase with only one bent. Precast I-girders are not chosen since they are unable to span the required length with the sections available from local precast manufacturers.

Box girder depth ≅ .08 x (70-2.5) = 5.4 ft.

Use (4) 5'-6" deep box girders with nominal width = 3'-6" which are available from local precast manufacturers.

The conceptual superstructure configuration is shown below.



TYPICAL SUPERSTRUCTURE SECTION

- Substructure configuration: (AREMA 9-1.4.3.3)

PREFERRED SUBSTRUCTURE	SPECIAL CONSIDERATION
✔Wide seats	Narrow seats
✔Seat bent caps	Integral bent caps
✔Multiple column	Single column

- Wide seats will be used at the abutments in accordance with AREMA 9-1.4.7.4.1

$N = (12 + 0.03L + 0.12H)(1 + 0.000125S^2)$ inches

L = length (ft) of the bridge deck to the adjacent movement joint.
S = angle of skew (degrees) measured from a line normal to the span.
H = At abutments, H is the average height (ft) of piers
At piers, H is the pier height (ft).

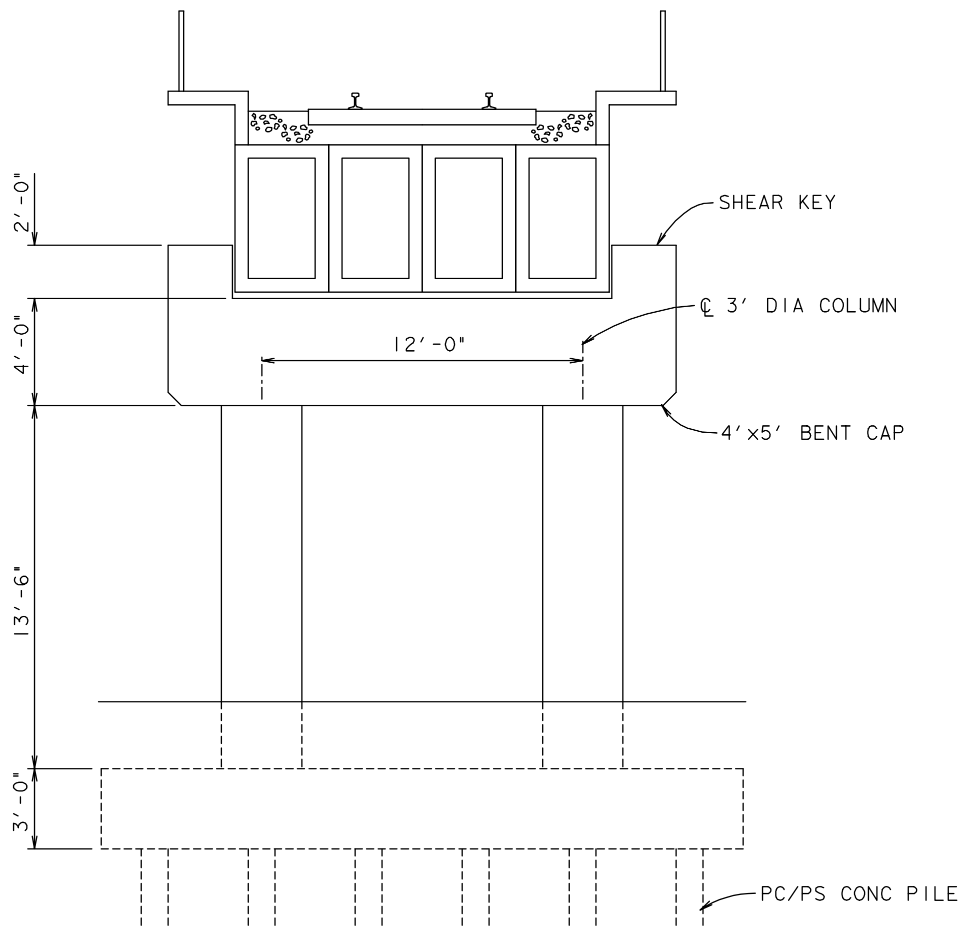
$N = [12 + 0.03(70) + 0.12(18)](1) = 16.3$ inches
Use minimum abutment seat width = 30" per standard practice in California.

- Multiple (2) column, seat-type bent will be used

  3'-0" diameter columns will be assumed. Circular columns will allow for efficient placement of spiral confinement reinforcement. The column diameter should be large enough to carry the service loads without significant slenderness effects but small enough to minimize the column plastic hinging load demands on the foundation.

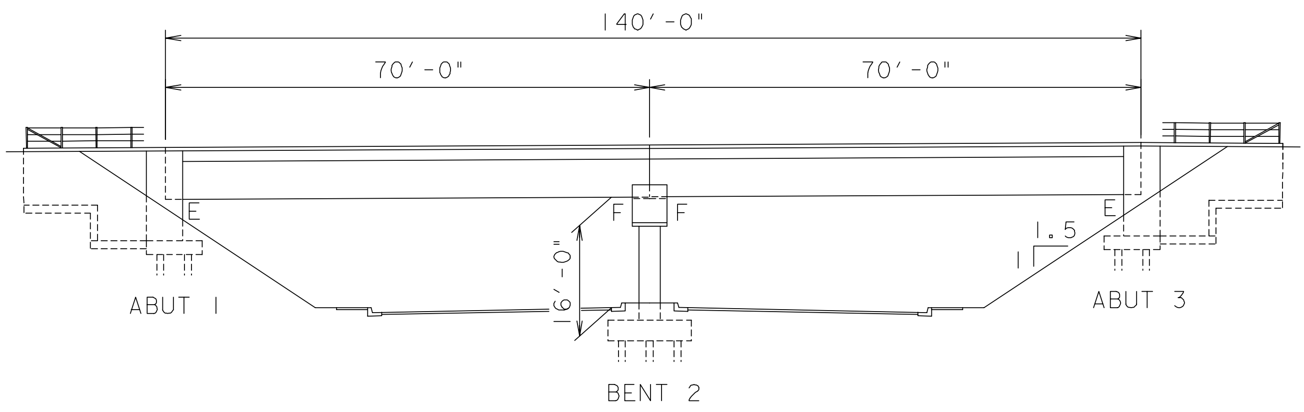
  5'-0" wide x 4'-0" deep bent cap will be assumed. The depth should be greater than or equal to the column size and the width is consistent with the 30" bearing seat width at the abutments and allows for reinforcing placement at the column joint.

  3'-0" deep, pile supported footing will be assumed.



BENT ELEVATION

The conceptual bridge configuration is shown below.



ELEVATION

DESIGN CRITERIA

Develop design criteria for the selected bridge concept.

Design specifications:

AREMA Manual for Railway Engineering

- Chapter 8 - Concrete Structures and Foundations
- Chapter 9 - Seismic Design for Railway Structures
- Chapter 19 - Bridge Bearings

Design references:

Caltrans Seismic Design Criteria: www.dot.ca.gov/hq/esc/earthquake_engineering/

Material properties:

Reinforced concrete

- Concrete compressive strength = f'_c = 4000 psi
- Reinforcing yield strength = f_y = 60,000 psi

Prestressed concrete

- Concrete compressive strength = f'_c = 5000 psi
- Concrete strength at transfer = f'_{ci} = 4,000 psi
- Prestressing steel tensile strength = f_{pu} = 270,000 psi (low-lax)

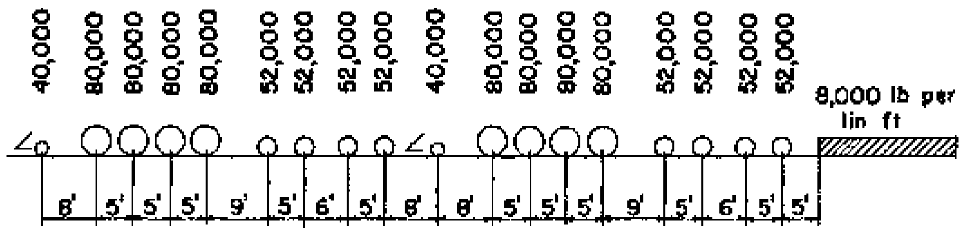
Soil parameters:

Soil is silty sand with a friction angle of 32.5 degrees and the water table is greater than 100 feet below ground and the bedrock depth exceeds 200 feet.

- Unit weight = 120 pcf
- Active lateral pressure = (1 - sin 32.5) / (1 + sin 32.5) x 120 = 36 pcf

Loads:

- Dead load should allow for an additional 6" of ballast depth.
- Live load is Cooper E80



Cooper E80 Load

Seismic:

Structure Importance Classification (AREMA 9-1.3.3.2)

1. Immediate Safety: (AREMA 9-1.3.3.2.1)

   - Occupancy Factor = 1 (No passenger service)
   - Hazardous Material Factor = 0.25 x 4 = 1 (25% hazardous material)
   - Community Life Lines Factor = 0 (Not crossing a community life line)

   Immediate Safety Factor = 1 + 1 = 2

2. Immediate Value: (AREMA 9-1.3.3.2.2)

   - Railroad Utilization Factor = 2 (12 million gross tons annual traffic)
   - Detour Availability Factor = 1 (No detour available)

   Immediate Value Factor = 2 x 1 = 2

3. Replacement Value: (AREMA 9-1.3.3.2.3)

   - Span Length Factor = 2 (70 feet)
   - Bridge Length Factor = 1.5 (140 feet)
   - Bridge Height Factor = 1 (23.5 feet)

   Replacement Value Factor = 2 x 1.5 x 1 = 3

Importance Classification Factor (AREMA 9-1.3.3.2.4)

- Serviceability = 0.8 x 2 + 0.2 x 2 + 0 x 3 = 2.0
- Ultimate = 0.1 x 2 + 0.8 x 2 + 0.1 x 3 = 2.1
- Survivability = 0 x 2 + 0.2 x 2 + 0.8 x 3 = 2.8

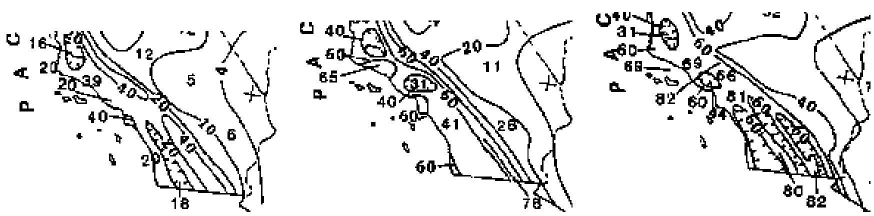
Return Periods:

Level 1 = 50 + 2(100-50) / 4 = 75 years
Level 2 = 200 + 2.1(500-200) / 4 = 358 years
Level 3 = 1000 + 2.8(2400-1000) / 4 = 1980 years

Base acceleration coefficients (AREMA 9-1.3.3.3)

- Determine base acceleration coefficients from AREMA maps for 100, 475 and 2400 year return periods.

  A(100) = 0.25G
  A(475) = 0.50G
  A(2400) = 0.60G



100 year 475 year 2400 year

- Determine base acceleration coefficients for calculated return periods.

  Note: FEMA 273, "NEHRP Guidelines for the Seismic Rehabilitation of Buildings", Section 2.6.1.3 uses natural logarithmic interpolation to determine accelerations for return periods between 475 and 2400 years. FEMA 273 uses an exponential function to determine the acceleration for return periods less than 475 years. The FEMA 273 procedures will be used to determine the base acceleration coefficients for the calculated return periods of 75, 358 and 1980 years.

- Return period = 75 years

$$A_{75} = A_{475}\left(\frac{P_R}{475}\right)^n$$

Since the 100 year return period is known, the exponent can be determined more precisely than the values given in the FEMA 273 table.

$$n = \frac{\ln\left(\frac{0.25}{0.50}\right)}{-1.558} = 0.445$$

$$A_{75} = 0.50\left(\frac{75}{475}\right)^{0.445} = 0.22$$

- Return period = 358 years

$$A_{358} = 0.50\left(\frac{358}{475}\right)^{0.445} = 0.44$$

- Return period = 1980 years

$$A_{1980} = e^x$$

$$x = \ln(A_{475}) + [\ln(A_{2400}) - \ln(A_{475})][0.606\ln(P_R) - 3.73]$$

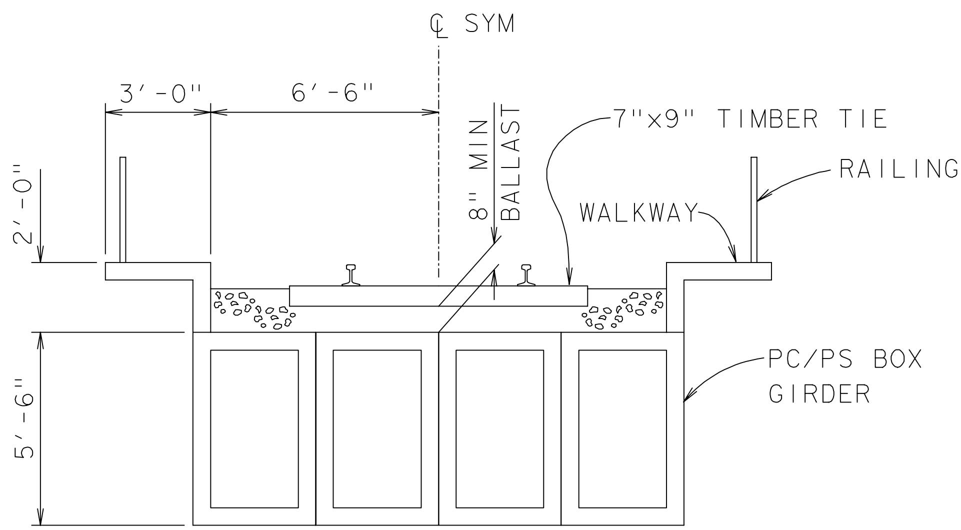
$$x = \ln(0.50) + [\ln(0.60) - \ln(0.50)][0.606\ln(1980) - 3.73] = -0.535$$

$$A_{1980} = e^{-0.535} = 0.59$$

SUPERSTRUCTURE DESIGN

Box Girder Design:

Use simple span length = 70 - 2.5 = 67.5 feet (c/c bearings)



TYPICAL SUPERSTRUCTURE SECTION

Assume constant PC/PS box girder thickness = 7 inches
Assume 1/2" gap between girders to be grouted solid.
Assume 7" thick walkway and ballast retainer.

- Dead load

  Box girders = 0.15(14.17 x 5.5 - 9.375 x 4.33) = 5.600 kips/ft
  Walkway/retainer = 0.15 x 2 x 0.583(3 + 1.42) = 0.773 kips/ft
  Ballast = 0.12(1.17 x 13 + 0.5 x 4.5) = 2.095 kips/ft
  Ties = 0.06 x 8.5 x 0.583 x 0.75 x 12 / 19.5 = 0.137 kips/ft
  Track = Use 0.2 kips/ft

  Total dead load = 8.805 kips/ft (Use 8.9 kips/ft)

  Girder dead load moment = $5.6(67.5)^2$ / 8 = 3189 kip-ft
  Girder dead load shear = 5.6(67.5) / 2 = 189 kips

  Additional dead load moment = $3.3(67.5)^2$ / 8 = 1879 kip-ft
  Additional dead load shear = 3.3(67.5) / 2 = 111 kips
  Total dead load pier reaction = 8.9 x 70 + 0.15(9.375 x 4.33 x 5) = 653 kips

- Live load (AREMA 15-Table 1-17)

Maximum Moments, Shears and Pier (or Floorbeam) Reactions for Cooper E80 Live Load or Alternative Live Load (Continued). All Values are for one rail (one-half track load). Interpolate for 67.5 foot span length.

SPAN LENGTH (FT)	MAXIMUM MOMENT (KIP-FT)	MAXIMUM MOMENT 1/4 POINT (KIP-FT)	MAXIMUM SHEAR (KIPS)			MAXIMUM PIER REACTION (KIPS)
			AT END	AT 1/4 POINT	AT CENTER	
60	2597.80	2010.00	196.00	120.21 (123.33)	55.69 (73.33)	306.42
70	3415.00	2608.20	221.04	131.89	61.45 (77.14)	354.08

Maximum live load moment = 2[2598 + 0.75(3415 - 2598)] = 6422 kip-ft
Maximum live load shear = 2[196 + 0.75(221 - 196)] = 430 kips
Maximum live load pier reaction = 2[306 + 0.75(354 - 306)] = 684 kips

- Impact (AREMA 8-17.2.3)

I = 14 + 800 / (67.5 - 2) = 26%

- Load combinations (AREMA 8-2.2.4)

- Service load (Group I): D + L + I

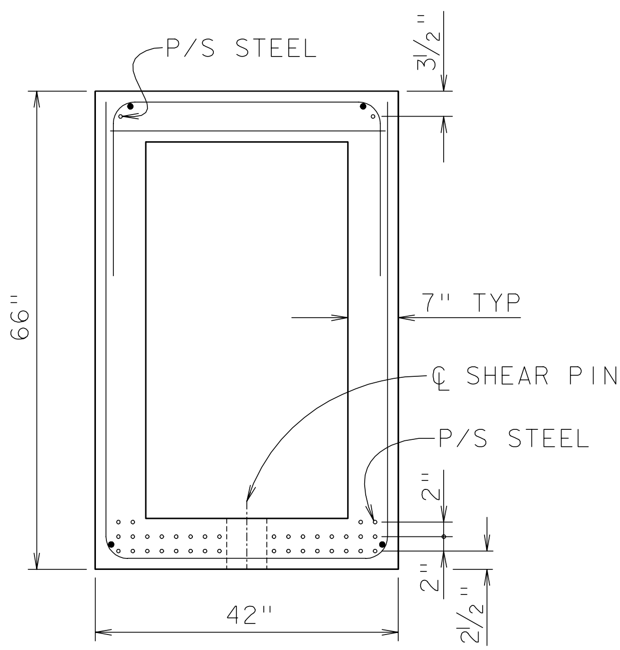
Moment = 5068 + 6422 + 0.26(6422) = 13160 kip-ft
Shear = 300 + 430 + 0.26(430) = 842 kips
Pier reaction = 653 + 684 + 0.26(684) = 1515 kips

- Load factor (Group I): 1.4(D + 5/3(L + I))

Moment = 1.4(5068 + 1.67 x 8092) = 26014 kip-ft
Shear = 1.4(300 + 1.67 x 542) = 1687 kips
Pier reaction = 1.4(653 + 1.67 x 862) = 2930 kips

- Design Prestressing Steel:

Use 1/2" diameter (A_s = 0.153 in^2), 270 ksi, low-lax prestressing steel.
Assume the P/S steel arrangement shown below.



BOX GIRDER SECTION

- Beam properties:

Area = 1316
I_x = 678151
I_Y = 312359
Y_{top} = Y_{bot} = 33
Y_{side} = 21

- Eccentricity:

[32(3.5)+4(6.5)+2(62.5)]/38=6.92"
e = 33 - 6.92 = 26.08"

- Stresses (AREMA 8-17.6.4)

Tension at bottom = f_t = 13160 x 12 x 33 / (678151 x 4) = 1.921 ksi
Compression on top = f_c = -1.921 ksi
Maximum tension allowed = 0 ksi

- Required prestress force after all losses:

P = 1.921 / (1 / 1316 + 26.08 x 33 / 678151) = 947 kips

Calculations using the PSBEAM program (developed by the author) are shown below and indicate that the assumed prestressing steel arrangement will satisfy the required prestressing force after all losses. Some minimal mild steel (#7 bar at each stirrup corner) is required to satisfy the ultimate moment requirements. Additional girder design calculations are required to determine the extent of strand debonding and shear reinforcement, however, they are not included in this example.

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   P S B E A M             *
*                                                   *
*                 INPUT DATA ECHO                   *
*                                                   *
* * * * * * * * * * * * * * * * * * * * * * * * * * *
1/05/01, 8:44 am

P R O G R A M   O P T I O N S
=====================================================

Units              = English (inches, pounds)
Design Criteria    = AREA
Prestressing       = Pretensioning
Section Properties = Gross

M A T E R I A L   P R O P E R T I E S
=====================================================

BEAM CONCRETE

     Unit weight                             = 0.0868
     Compressive strength at 28 days         = 5000.00
     Compressive strength at prestressing    = 4000.00
     Modulus of elasticity at 28 days        = 4286415.
     Modulus of elasticity at prestressing   = 3833886.

MILD REINFORCING

     Yield strength                          = 60000.00
     Modulus of elasticity                   = 29000000.

PRESTRESSING STEEL

     Type                                    = Low relaxation strand
     Diameter                                = 0.500
     Ultimate strength                       = 270000.00
     Yield strength                          = 243000.00
     Modulus of elasticity                   = 28000000.

S E C T I O N   P R O P E R T I E S
=====================================================

BEAM SECTION AND DIMENSIONS

     Box Girder
     User defined
     D    = 66.000
     Btop = 42.000
     Htop = 7.000
     Bweb = 7.000
     Bbot = 42.000
     Hbot = 7.000
     Ftop = 0.000
     Fbot = 0.000
```

```
BEAM PROPERTIES

     Area = 1316.00
     MoI  = 678151.
     Ytop = 33.000
     Ybot = 33.000

L O A D S
=====================================================

Initial prestress force  = 1177335.0
Number of load locations =  1

LOCATION NUMBER  1  AT X =  405

CASE                   AXIAL         SHEAR         MOMENT        FACTOR
-----------------------------------------------------------------------
BEAM DEAD              0.            0.            9567000.      1.400
NON-COMP DEAD          0.            0.            5637000.      1.400
COMPOSITE DEAD         0.            0.            0.            0.000
LIVE PLUS IMPACT       0.            0.            24276000.     2.330
PRESTRESS              0.            0.            0.            0.000
SECONDARY PRESTRESS    0.            0.            0.            0.000
MISCELLANEOUS          0.            0.            0.            0.000

A L L O W A B L E   S T R E S S   A N D   L O S S
=====================================================

ALLOWABLE STRESS

     Initial concrete compressive stress = 2400.
     Initial concrete tensile stress     = 474.
     Final concrete compressive stress   = 2000.
     Final concrete tensile stress       = 0.
     Initial prestress                   = 202500.

PRESTRESS LOSS

     Total prestress loss                = 38424.
     Prestress loss at time of transfer  = 12979.
     Anchor set                          = 0.000

C O N F I G U R A T I O N
=====================================================

PRESTRESS PATH

     Straight
     Xleft       = 0.0               Yleft   = 0.000
     Xmiddle     = 405.0             Ymiddle = 6.920
     Xright      = 0.0               Yright  = 0.000

BEAM CONFIGURATION

     Beam length         = 810.000
     Beam spacing        = 42.000
```

```
* * * * * * * * * * * * * * * * * * * * * * * * * * *
*                                                   *
*           P R O G R A M   P S B E A M             *
*                                                   *
*                   OUTPUT DATA                     *
*                                                   *
* * * * * * * * * * * * * * * * * * * * * * * * * * *

P R E S T R E S S   F O R C E S
=====================================================

LOCATION NUMBER  1  AT X =  405

     CASE                            VALUE
     --------------------------------------------
     INITIAL FORCE                   1177335.
     CABLE ECCENTRICITY              -26.08
     FORCE AT TRANSFER               1101876.
     MOMENT AT TRANSFER              -28736912.
     FINAL FORCE                     953939.
     FINAL MOMENT                    -24878738.

B E A M   S T R E S S E S
=====================================================

LOCATION NUMBER  1  AT X =  405
Prestress loss =  38424

     CASE                            TOP FIBER     BOTTOM FIBER
     ----------------------------------------------------------
     INITIAL PRESTRESS               -561.         2236.
     FINAL PRESTRESS                 -486.         1936.
     SECONDARY PRESTRESS             0.            0.
     BEAM DEAD                       466.          -466.
     NON-COMP DEAD                   274.          -274.
     COMPOSITE DEAD                  0.            0.
     LIVE PLUS IMPACT                1181.         -1181.
     ----------------------------------------------------------
     INIT P/S + BEAM DL              -96.          1770.
     FINAL P/S + TOT DL              254.          1196.
     P/S + ALL LOADS                 1435.         14.

CHECK CRITICAL CONCRETE STRESSES

     CASE                            ACTUAL        ALLOWABLE
     ----------------------------------------------------------
     INITIAL TENSION                 -96.          -474.         < OK >
     INITIAL COMPRESSION             1770.         2400.         < OK >
     FINAL TENSION (DL ONLY)         1196.         0.            < OK >
     FINAL TENSION                   14.           0.            < OK >
     FINAL COMPRESSION               1435.         2000.         < OK >
```

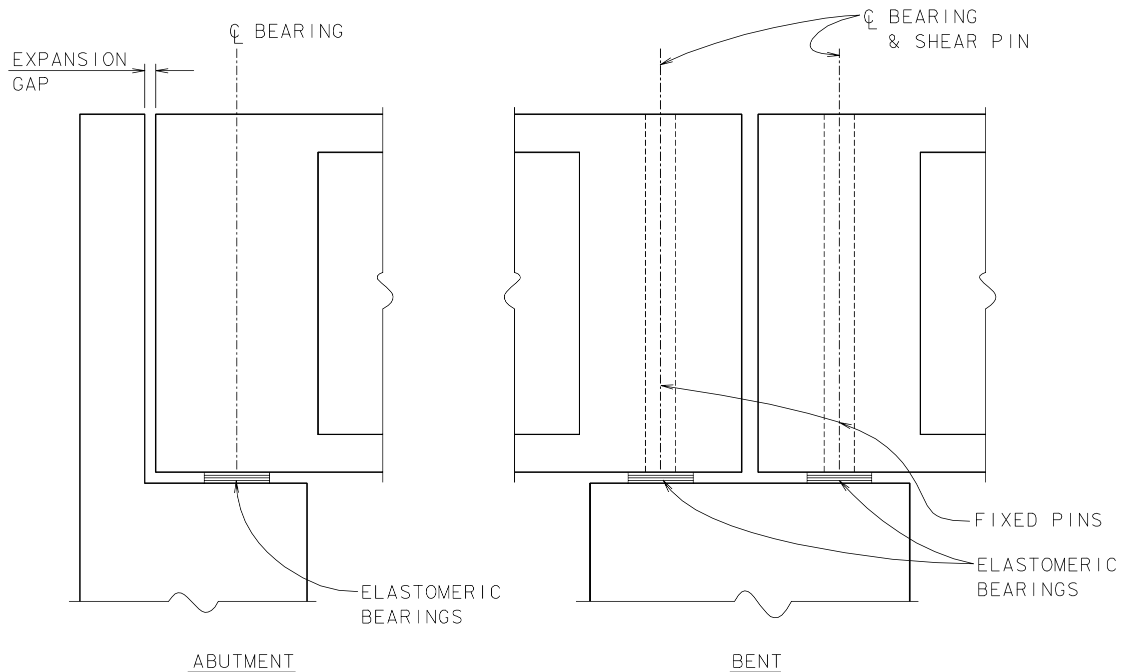
```
M O M E N T   C A P A C I T Y
=====================================================

X        Mult          1.2Mcr        phiMn
-----------------------------------------------------
 405     77848680.     13077953.     76452936.     Mild steel = 1.275

B E A M   D E F L E C T I O N S
=====================================================

Initial prestress deflection     =   0.906
Final prestress deflection       =   0.702
Simple beam dead load deflection =   0.220

S H E A R   C A P A C I T Y
=====================================================

X        Vult          phiVn         Av/S
-----------------------------------------------------
 405     0.            89484.        0.012
```

Bearing Design:

Design elastomeric bearings at abutments and bent.
Assume 2 bearings per girder (total 8).

- Movements (AREMA 19-1.1.2a)

  Service load movement = 1" x 70 / 100 = 0.70 inch
  Use 3/4" expansion gap at abutments.



SECTION AT ELASTOMERIC BEARINGS

- Loads

  Dead load = [189 + 111 + 0.15(9.375 x 4.33 x 2.5)] / 8 = 39.4 kips
  Live load plus impact = 430 x 1.26 / 8 = 67.7 kips

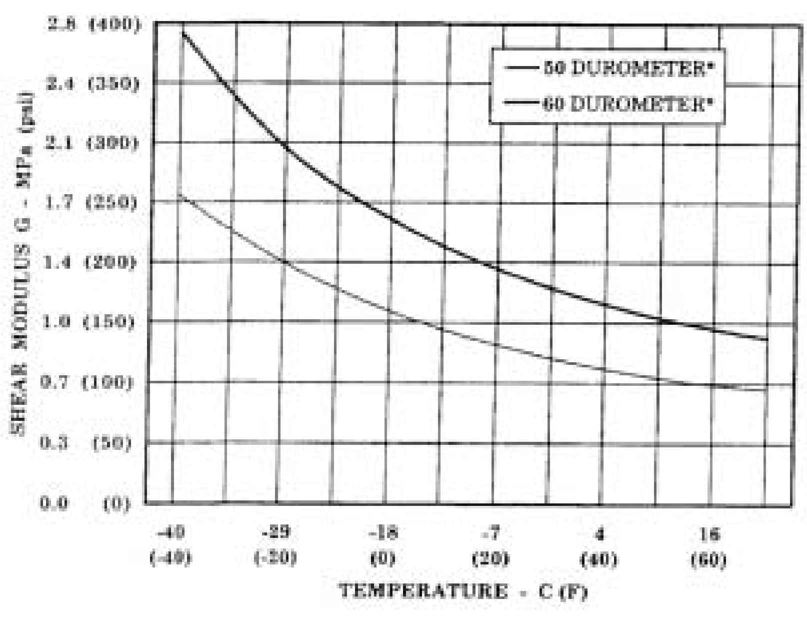
- Design

- Compressive stress (AREMA 19-1.6.3.4)

Compressive stress = 1000 psi for steel reinforced bearings
Area = A > (39.4 + 67.7) / 1.0 = 107 in^2
Square bearing width > $(107)^{1/2}$ = 10.3"

Try 12" x 12" bearing (A = 144 in^2)

Compressive stress = f_a = 107.1 / 144 = 0.74 ksi < G S / k
Shear modulus = G = 140 psi (Figure 1-1c at 70 degrees F)
Shape factor = S = L x W / (2 x t_i(L + W))
S = 12 x 12 / (2 x 0.5 (12 + 12)) = 6
Modifying factor = k = 1.0
f_a = 140 x 6 / 1.0 = 840 psi > 740 psi   Okay



Relationship of shear modulus to hardness of neoprene compounds at various temperatures

Graph C

- Shear (AREMA 19-1.6.3.7)

Bearing thickness = T > $2d_s$
Design for d_s = 3/4" to close gap at abutment during Level 1 seismic event

T = 2 x 3/4 = 1.5"   Try 2" thick (nominal) bearing

Shear force = $F_s = G\ d_s\ A / T$
G = 180 psi (Figure 1-1c at 30 degrees F)
F_s = 180 x 3/4 x 144 / 2000 = 9.72 kips/bearing

- Anchorage (AREMA 19-1.6.3.10)

The bearing must be secured against horizontal movement if the shear force exceeds 20% of the vertical load.

0.2 x 39.4 = 7.88 kips < 9.72 kips

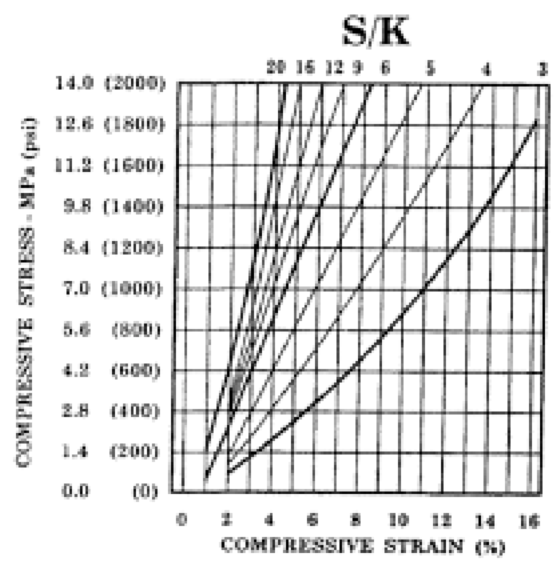
Therefore, the bearings must either be thicker to reduce the shear force or be secured against horizontal movement.

Required thickness to eliminate anchorage = T = 2 x 9.72 / 7.88 = 2.47"
Try using 2.5" thick (nominal) bearing to reduce loading

- Need to check stability (AREMA 19-1.6.3.8) and compressive deflection (AREMA 19-1.6.3.5) for thicker bearing pads.

Stability is okay if T < L / 3 = 12 / 3 = 4"

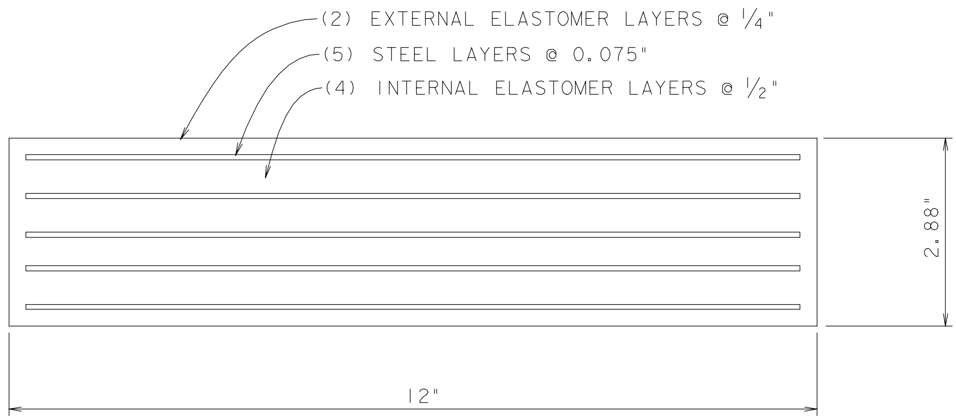
Comp deflection = $d_c = \Sigma e_c t$ < 1/8"
LL+I stress = 67.7 / 144 = 0.47 ksi
Comp strain = e_c = 0.025 (Figure 1-1a)
d_c = 0.025 x 2.5 = 0.063" < 0.125" Okay



Compressive Stress/Strain of Steel Reinforced Neoprene Bearings (hardness of neoprene compound - 60 Durometer A)

Graph A

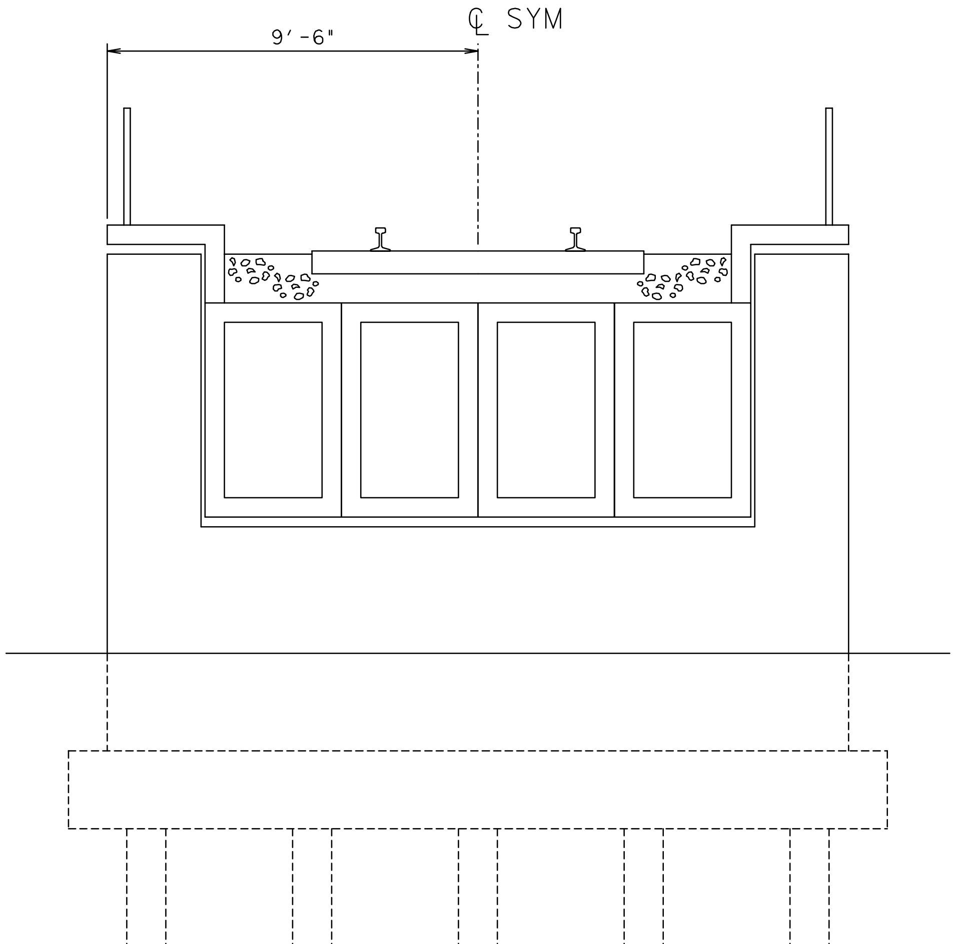
Final bearing pad size = 2.5" (nominal) x 12" x 12" steel reinforced bearing



STEEL REINFORCED ELASTOMERIC BEARING

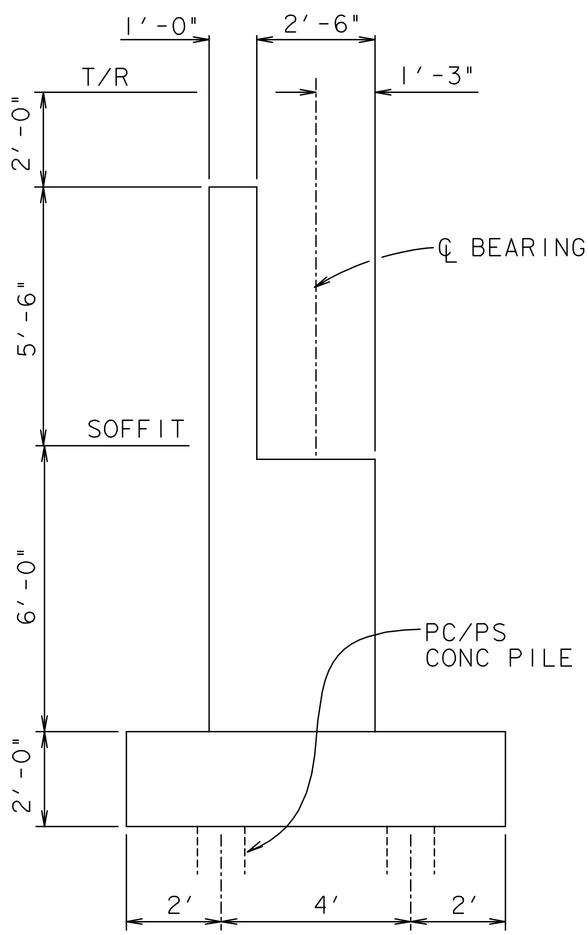
ABUTMENT DESIGN

Size the abutment for static loads.



ABUTMENT ELEVATION

Abutment Loads (AREMA 8-2.2.3)



ABUTMENT SECTION

- Dead load

  P_v = 189 + 111 = 300 kips (Pg C-1)

  w1 = 1 x 7.5 x 19 x 0.15 = 21.4 kips
  w2 = 3.5 x 6 x 19 x 0.15 = 59.9 kips
  w3 = 8 x 2 x 21 x 0.15 = 50.4 kips
  w4 = 1.75 x 13.5 x 2 x 0.15 = 7.1 kips

  Total = 439 kips

- Live load

  P_v = 430 kips (Pg C-2)

- Longitudinal force

  See sheet F-2 for longitudinal force calculations.

- Earth pressure

  $P_e = 0.036(19)(15.5)^2/2 = 82.2$ kips
  $P_{sc} = 0.3(16)(15.5) = 74.4$ kips

  Total = 157 kips

- Service loads

  M_{ot} = 82.2 x 15.5 / 3 + 74.4 x 15.5 / 2 = 1001 k-ft
  M_{rt} = (300 + 430) x 4 + 21.4 x 5.75 + 59.9 x 4.5 + 50.4 x 4 + 7.1 x 7.1 = 3565 k-ft

  x = (3565 - 1001) / 869 = 2.95 ft.
  eccentricity = 4 - 2.95 = 1.05 ft.

- Pile Design

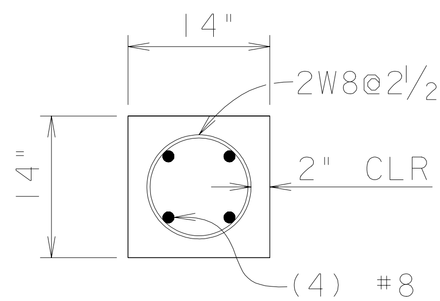
Section properties:

Pile area = 196
Pile moment of inertia = 3201
Pile group section modulus = $5(2)^2 = 20$

Lateral load on piles = 157 / 10 = 15.7 kips/pile

P_{max} = 869 / 10 + 869 x 1.05 / 20 = 133 kips
P_{min} = 869 / 10 - 869 x 1.05 / 20 = 41 kips

Use (10) 14" square PC/PS concrete piles.



TYPICAL PILE SECTION

- Shear capacity (AREMA 8-35)

For spiral reinforcing: $\phi V_s = \phi A_v f_y D' \pi / (2 \times s)$

As = 0.08 in^2
ϕ = 0.319"
D' = 14 - 4 - 0.319 = 9.68"
ϕV_s = 0.85 x 0.08 x 60 x 9.68 x 3.1416 / (2 x 1.25) = 49.6 kips

- Lateral pile analysis

Perform lateral load analysis with LPILE program to determine bending moment and stiffness.

Soil is sand with friction angle ϕ = 32.5°
Assume the piles are pinned at the top.
Assume soil modulus k = 90 pci (LPILE users manual table 3.2)

Analysis results:

Load (kips)	Deflection (inches)	Moment (k-ft)	Stiffness (k/in)
10	0.066	18	151
20	0.147	40	136
30	0.258	67	117
40	0.410	101	98
50	0.614	142	81

- Interaction diagram

c	Mn	Pn	Comments
	0	-189.6	Maximum tension
0.7	23	-148.8	
1.4	44.3	-108	
2.1	71.4	-45.3	
2.58	89.8	0.2	Pure bending
2.8	97.5	19.1	
3.07	106.1	41.1	Load condition no 1
3.5	118.7	74.1	
4.2	134.9	122.8	
4.29	137	133.8	Load condition no 2
4.9	150.2	203	
5.6	163.3	273.5	
6.3	174.3	337.3	
6.59	178.2	362.5	Balanced strain
7	180.9	403.5	
7.7	184.7	469.3	
8.4	187.3	530.9	
9.1	188.4	589.4	
9.8	187.8	644.2	
10.5	185.7	687.3	
11.2	182.1	738.4	
11.9	177	788.3	
12.6	170.5	837.2	
13.3	162.3	885.2	
	0	1136.4	Maximum compression

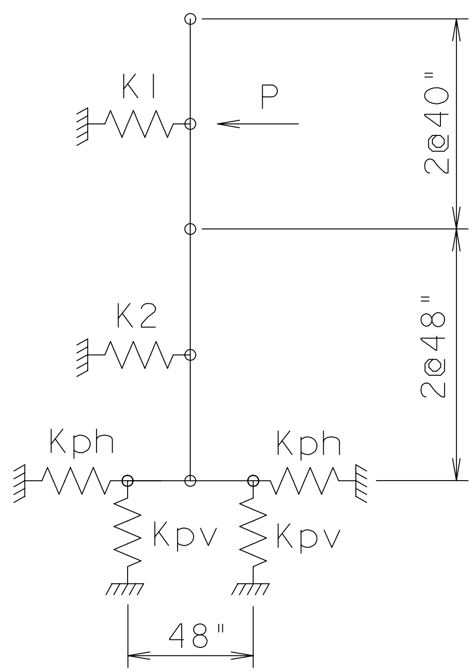
Dead plus live load combination is not critical lateral load for pile design.

- Additional abutment design calculations are required to determine the abutment wall and footing reinforcement, however, they are not included in this example.

Abutment longitudinal stiffness

The abutment longitudinal stiffness will be determined for use in the seismic analysis. There has been considerable research, both empirical and analytical, on the capacity and stiffness of the abutment for highway bridges. At this time, however, there has been no consensus as to what abutment capacity and stiffness is appropriate. Caltrans uses a capacity based on a uniform soil passive resistance of 7.7 ksf for an 8' deep superstructure, but this assumes that the abutment breaks off during a maximum credible earthquake event. The longitudinal capacity and stiffness of the soil behind the abutments for this railroad bridge will be engaged once the 3/4" expansion gap is closed at the abutment, however, the backwall cannot break off and still satisfy the Level 1 performance requirements. There will also be some tensile capacity of the CWR track at the opposite abutment.

- The abutment stiffness will be calculated using a simple finite element model to represent the stiffness of the soil and piles.



ABUTMENT MODEL

- The stiffness of the soil behind the abutment will be simulated with two soil springs generated using the following assumptions.

    1. Soil friction angle ϕ = 32.5 degrees
    2. Dynamic passive pressure = 1.5 x Coulomb passive pressure
    3. Passive pressure fully develops with a movement of 2% of the wall height

$$K_p = \frac{\sin^2(\alpha - \phi)}{\sin^2\alpha \sin(\alpha + \delta)\left[1 - \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi + \beta)}{\sin(\alpha + \delta)\sin(\alpha + \beta)}}\right]^2}$$

$\alpha = 90°$
$\beta = 0°$
$\delta = 2 \times 32.5 / 3 = 21.7°$

$K_p = 7.7$

Capacity P1 = 7.7 x 1.5 x 0.12 x 3.33 x (6.67 x 19) = 585 kips
Capacity P2 = 7.7 x 1.5 x 0.12 x 10.67 x (8 x 19) = 2248 kips
Deflection = 0.02 x 14.67 x 12 = 3.52"

K1 = 585 / 3.52 = 166 k/in
K2 = 2248 / 3.52 = 639 k/in

- The following pile stiffnesses will be assumed.

For the vertical pile stiffness, assume that the pile has a service load capacity of 70 tons and that this value was developed from a yield load capacity of 140 tons at a 1/2" deflection.

K_{pv} = 140 x 2 x 5 / 0.5" = 2800 k/in

For the (average) lateral pile stiffness, a review of the stiffness table and the interaction diagram on page D-4 shows that a reasonable average yield bending moment is 100 k-ft, which corresponds to a stiffness of approximately 100 k/in, therefore a reasonable average lateral pile stiffness is:

K_{ph} = 100 kips/in x 5 = 500 k/in

- Finite element model results (File = abut.sdb)

  Load = P = 100 kips
  Lateral deflection = 0.252 inches
  Abutment stiffness = K = 100 / 0.252 = 397 k/in

  P1 = 41.8 kips
  P2 = 54.6 kips
  Pph = 1.8 kips
  Ppv = +/- 110 kips
  Shear in backwall = V_u = 100 - 41.8 = 58.2 kips

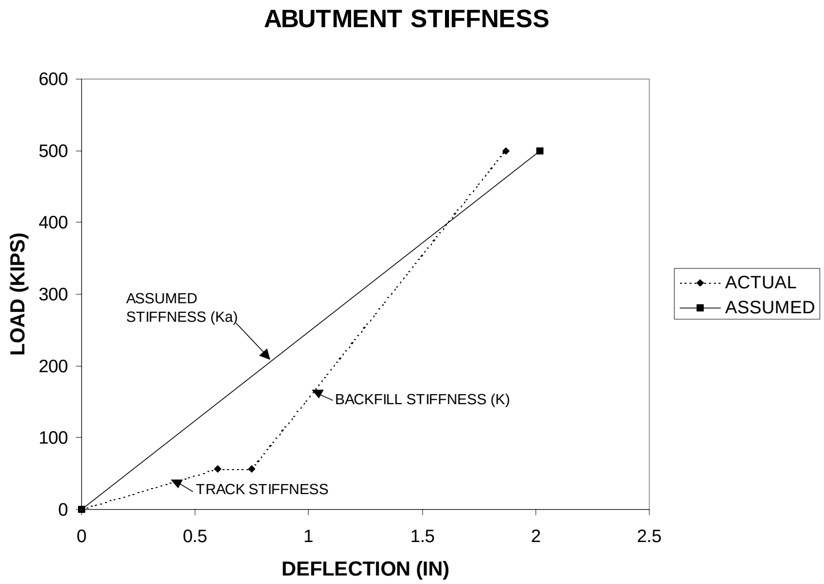
  Abutment capacity will be controlled by shear capacity of backwall:

  $\phi V_c = 0.85 \times 2\ (4000)^{1/2} \times 19 \times 12 \times 9.5 / 1000 = 233$ kips
  P = 100 x 233 / 58.2 = 400 kips

- Abutment stiffness

  Assume 2" maximum superstructure movement.
  Initial abutment stiffness = K_a = 397(2 - 0.75) / 2 = 248 k/in (include 3/4" gap)

  The actual abutment stiffness is shown below for comparison. The actual abutment stiffness consists of the track stiffness with the soil stiffness developed after the 3/4" expansion gap is closed.



The Caltrans abutment capacity for comparison purposes is 7.7 x $(6.67)^2$ x 19 / 8 = 814 kips at 2% abutment movement = 0.02 x 176 = 3.52" for an initial stiffness of 814 / 3.52 = 231 k/in.

SEISMIC ANALYSIS

Analyze structure for Level 1 ground motion.

Analysis procedure selection (AREMA 9-1.4.4.2)

Two-span bridge can use Equivalent Lateral Force (ELF) procedure.

Structure response (AREMA 9-1.4.3)

- Site coefficient (AREMA 9-1.4.3.1)

From page B-1: Soil is silty sand with a friction angle of 32.5 degrees. The water table is greater than 100 feet below ground and the bedrock depth exceeds 200 feet.

Table 1.5 Site Coefficient

Soil Type	Description	Site Coefficient
1	Rock of any characteristic, either shale-like or crystalline in nature, that may be characterized by a shear wave velocity greater than 2,500 feet per second, or stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying the rock are stable deposits of sand, gravel, or stiff clays.	1.0
2	Deep cohesionless or stiff clay conditions where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravel, or stiff clays.	1.2
3	20 to 40 feet of soft to medium-stiff clays with or without intervening layers of cohesionless soils.	1.5
4	Soil containing more than 40 feet of soft clays or silts, that may be characterized by a shear wave velocity of less than 500 feet per second.	2.0

The soil corresponds to type 2 with a site coefficient, S = 1.2

- Damping adjustment factor (AREMA 9-1.4.3.2)

$$D = \frac{1.5}{(0.4\xi + 1)} + 0.5$$

ξ = Percent critical damping

Assume damping increase to 10% in longitudinal direction due to the presence of CWR on this short (less than 300' long) bridge (AREMA C-1.4.7.4.5).

$$D = \frac{1.5}{(0.4(10) + 1)} + 0.5 = 0.8$$

- Seismic response coefficient (AREMA 9-1.4.3.3)

$$C_m = \frac{1.2ASD}{T_m^{2/3}} \leq 2.5AD$$

Base acceleration coefficient = A = 0.22G (Page B-4)

$$\text{Trans } C_{tm} = \frac{1.2(0.22)(1.2)1.0}{T_{tm}^{2/3}} \leq 2.5(0.22)(1.0) = \frac{0.317}{T_{tm}^{2/3}} \leq 0.55$$

$$\text{Long } C_{lm} = \frac{1.2(0.22)(1.2)0.8}{T_{lm}^{2/3}} \leq 2.5(0.22)(0.8) = \frac{0.253}{T_{lm}^{2/3}} \leq 0.44$$

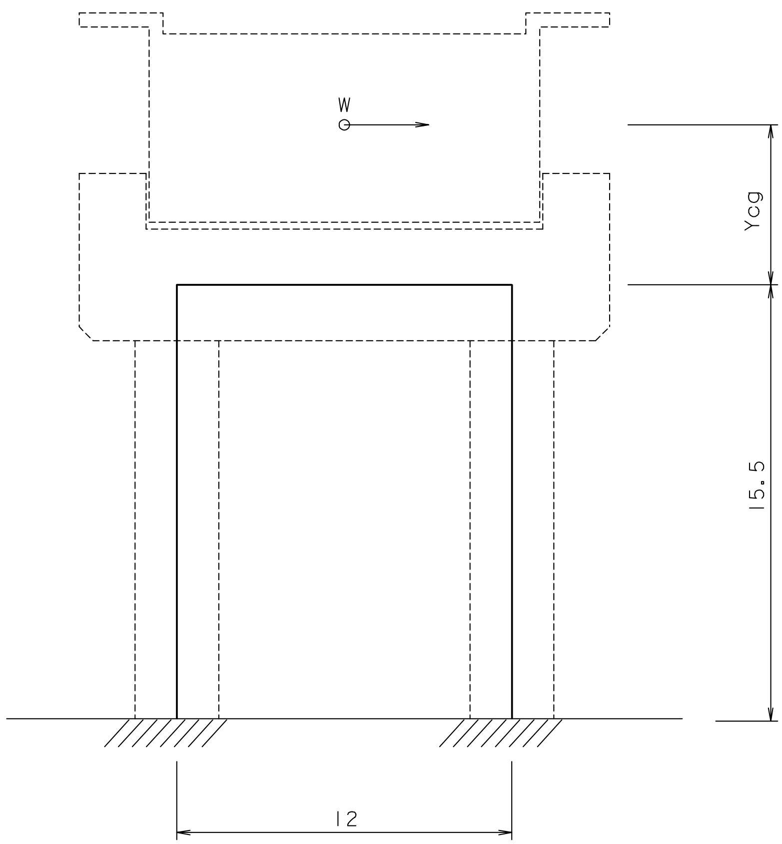
Equivalent Lateral Force Procedure (AREMA 9-1.4.4.3):

- Calculate the transverse natural period of vibration (T_{tm}) of the structure.

  Since the bridge superstructure is simply-supported, there is no coupling of modes between the abutments and bent. Therefore the transverse natural period of vibration will be calculated separately at the abutments and bent.

$$T_{tm} = 2\pi\sqrt{\frac{W}{gK}}$$

W = Total weight of the bridge (tributory)
g = Acceleration due to gravity (length/time^2) = 386 in/s^2
K = The total structure stiffness including the stiffness of the superstructure, supporting members and surrounding soil.



TRANSVERSE BENT MODEL

- Section properties:

Bent cap: $A = 5 \times 4 = 20\ ft^2$
$I_g = 5(4)^3 / 12 = 160\ ft^4$
Assume $I_e = 0.75 \times 160 = 120\ ft^4$

Column: $A = 3.1416(1.5)^2 = 7.07\ ft^2$
$I_g = 3.1416(1.5)^4 / 4 = 3.98\ ft^4$
Assume $I_e = 0.5 \times 3.98 = 1.99\ ft^4$

- Weight:

The weight consists of the tributory superstructure weight, the bent cap weight and 1/2 the columns weight.

W = 653 (page C-1) + 0.15 x (20 x 19 + 7.07 x 13.5) = 724 kips

Superstructure:

$$Y_{CG1} = 2 + \frac{5.6 \times 2.75 + 0.773 \times 6.9 + 2.095 \times 5.9 + 0.137 \times 6.5 + 0.2 \times 7.1}{8.805} = 4.0'$$

Bent cap and columns:

$$Y_{CG2} = \frac{57 \times 0 - 14.3 \times 5.375}{71.3} = -1.08'$$

$$Y_{CG} = \frac{653 \times 4 - 71.3 \times 1.08}{724} = 3.5'$$

- Stiffness:

1. Foundation stiffness can be determined from the individual pile stiffnesses and the pile group arrangement along with the passive soil stiffness at the pile cap.

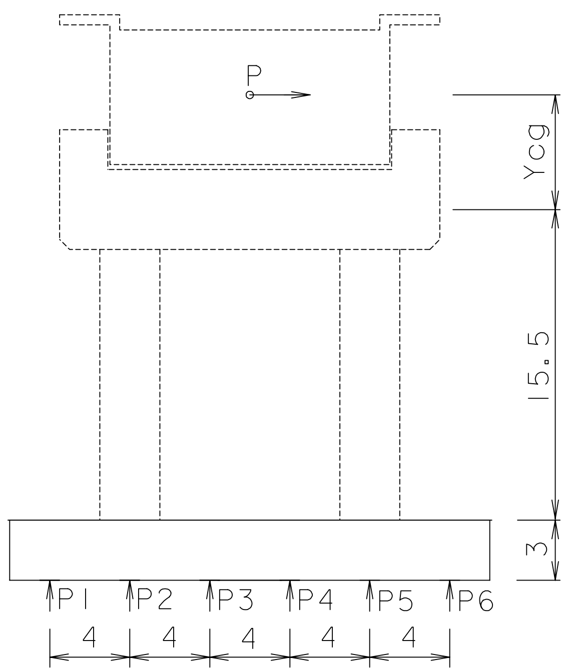
   A. For the vertical pile stiffness, assume that the pile has a service load capacity of 100 tons and that this value was developed from a yield load capacity of 200 tons at a 1/2" deflection.

      K_{pv} = 200 x 2 / 0.5" = 800 k/in (per pile)

   B. For the (average) lateral pile stiffness, a review of the stiffness table and the interaction diagram on page D-4 shows that a reasonable average yield bending moment is 100 k-ft, which corresponds to a stiffness of approximately 100 k/in, therefore a reasonable average lateral pile stiffness is:

      K_{ph} = 100 kips/in (per pile)
      K_{gh} = 100 x 18 = 1800 k/in (group)

   C. For the rotational pile group stiffness, a unit lateral deflection will be assumed at the superstructure CG and the individual pile loads will be calculated from the geometry.



BENT MODEL

θ = 1 / [(15.5 + 3.5 + 3) x 12] = 0.00379 radians

P1 = -P6 = -0.00379 x 10 x 12 x 800 = -364 kips
P2 = -P5 = -0.00379 x 6 x 12 x 800 = -218 kips
P3 = -P4 = -0.00379 x 2 x 12 x 800 = -73 kips

K_{pr} = (364 x 6 x 10 + 218 x 6 x 6 + 73 x 6 x 2) / 22 = 1389 k/in

D. The passive soil stiffness at the pile cap will be simulated with a soil spring generated using the following assumptions.

i. Soil friction angle ϕ= 32.5 degrees
ii. Dynamic passive pressure = 1.5 x Coulomb passive pressure
iii. Full passive pressure develops with a movement of 2% of the footing depth

$$K_p = \frac{\sin^2(\alpha - \phi)}{\sin^2\alpha \sin(\alpha + \delta)\left[1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi + \beta)}{\sin(\alpha + \delta)\sin(\alpha + \beta)}}\right]^2}$$

α = 90°
β= 0°
δ = 2 x 32.5 / 3 = 21.7°

K_p = 7.7

Capacity P1 = 7.7 x 1.5 x 0.12 x 3 x (3 x 12) = 150 kips
Deflection = 0.02 x 4.5 x 12 = 1.08"

K = 150 / 1.08 = 139 k/in

2. Structure stiffness can be determined by applying a unit horizontal force to a bent finite element model and determining the deflection. Note that the bent model does not include foundation flexibility.

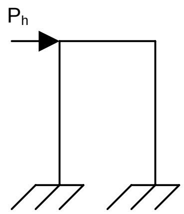
Unit load = 100 kips
Deflection = 0.202 inches (See Transverse Analysis of Bent, below)
K = 100 / 0.202 = 495 kips/in

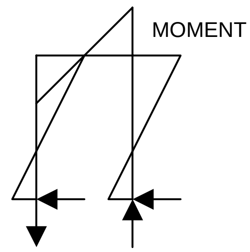
3. The total transverse bent stiffness can be determined by summing up the foundation stiffness and the structure stiffness.

$K = [(1800 + 139)^{-1} + (1389)^{-1} + (495)^{-1}]^{-1}$ = 307 k/in

- Transverse Analysis of Bent (File = bent.sdb):

1. The results of a transverse bent analysis are shown below for a unit lateral load of 100 kips.

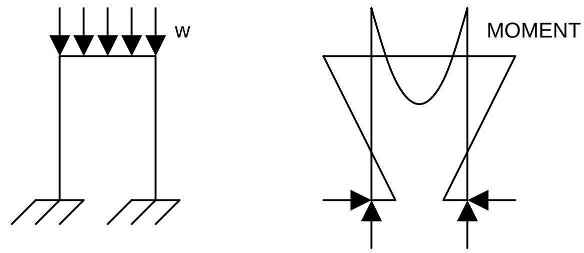




ANALYSIS RESULTS

LOAD	ITEM	BENT CAP	LEFT COLUMN	RIGHT COLUMN
P_h = 100 kips	AXIAL	-	-92 k	92 k
	SHEAR	63 k	50 k	50 k
	M+ve	377 k-ft	377 k-ft	397 k-ft
	M-ve	377 k-ft	397 k-ft	377 k-ft
	DEFL	0.202 in		

2. The results of a transverse bent analysis are shown below for dead load, live load and impact.



ANALYSIS RESULTS

LOAD	ITEM	BENT CAP	LEFT COLUMN	RIGHT COLUMN
DEAD LOAD (w = 653/14)	AXIAL	-	371 k	371 k
	SHEAR	298 k	7 k	7 k
	M+ve	780 k-ft	35 k-ft	35 k-ft
	M-ve	114 k-ft	72 k-ft	72 k-ft
LIVE LOAD (w = 684/14)	AXIAL	-	342 k	342 k
	SHEAR	293 k	7 k	7 k
	M+ve	782 k-ft	35 k-ft	35 k-ft
	M-ve	98 k-ft	73 k-ft	73 k-ft
IMPACT (w = 178/14)	AXIAL	-	89 k	89 k
	SHEAR	76 k	2 k	2 k
	M+ve	203 k-ft	9 k-ft	9 k-ft
	M-ve	25 k-ft	19 k-ft	19 k-ft

- Natural period:

$$T_{tm} = 2\pi\sqrt{\frac{724}{386(307)}} = 0.491\,\text{seconds}$$

- Calculate the Transverse Seismic Response Coefficient (C_m) for the structure (Page E-2).

$$\text{Transverse } C_{tm} = \frac{0.317}{0.491^{2/3}} = 0.51 < 0.55 \;\therefore \text{Use } 0.51$$

Since the abutment is stiffer than the bent, the transverse seismic response coefficient at the abutment will be controlled by the maximum response of 0.55Gs.

- Perform static analysis on the bridge in the transverse direction.

  - Calculate the distributed seismic load from the following formula.

    $p(x) = C_m\, w(x)$

    p(x) = distributed seismic load per unit length of bridge
    w(x) = distributed weight of bridge per unit length

    1. Uniform lateral load on spans:

       p(x) = 0.51 x 8.9 (page C-1) = 4.54 k/ft

    2. Lateral load on bent:

       P = 0.51(724) = 369 kips

    3. Lateral load on abutments:

       P = 0.55(653 / 2) = 180 kips

  - Distribute the seismic load to individual bent members based on the stiffness and support conditions.

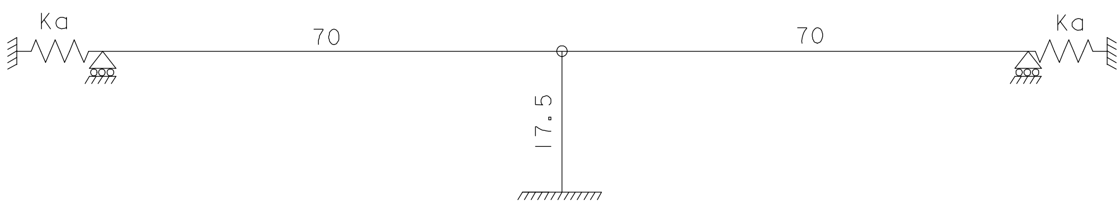
    Ratio unit lateral load results (page E-7)

BENT MEMBER LOADS

LOAD	ITEM	BENT CAP	LEFT COLUMN	RIGHT COLUMN
P_h = 369 kips	AXIAL	-	-339 k	339 k
	SHEAR	232 k	185 k	185 k
	M+ve	1391 k-ft	1391 k-ft	1465 k-ft
	M-ve	1391 k-ft	1465 k-ft	1391 k-ft
	DEFL	0.745 in		

- Calculate the longitudinal natural period of vibration (T_{lm}) of the structure.

  The natural period of the bridge in the longitudinal direction will be determined by considering the stiffness of the bent and one of the abutments compressing against the soil. The initial abutment stiffness is calculated on page D-7 and a final secant stiffness may need to be recomputed if the abutment capacity is exceeded in the original calculation iteration.



LONGITUDINAL BRIDGE MODEL

- Stiffness:

1. Longitudinal bent stiffness:

   A. Foundation stiffness can be determined from the individual pile stiffnesses and the pile group arrangement along with the passive soil stiffness at the pile cap.

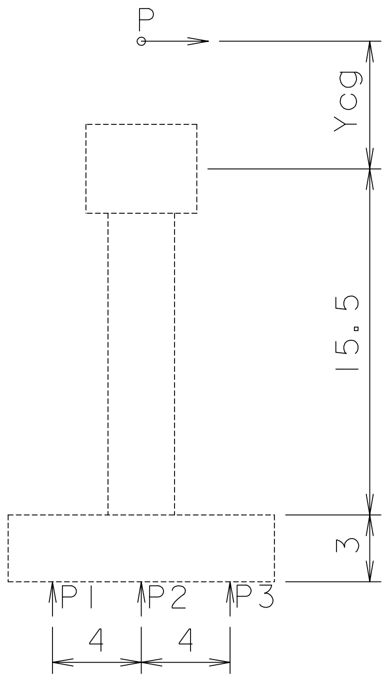
      i. Vertical pile stiffness (Page E-5)

         K_{pv} = 800 k/in (per pile)

      ii. Lateral pile stiffness (Page E-5)

         K_{gh} = 1800 k/in (group)

      iii. For the rotational pile stiffness, a unit lateral deflection will be assumed at the superstructure CG and the individual pile loads will be calculated from the geometry.



BENT MODEL

θ = 1 / [(15.5 + 3.5 + 3) x 12] = 0.00379 radians

P1 = -P3 = -0.00379 x 4 x 12 x 800 = -145 kips

K_{pr} = (145 x 12 x 4) / 22 = 316 k/in

iv. Passive soil stiffness at the pile cap (Page E-6)

Capacity P2 = 7.7 x 1.5 x 0.12 x 3 x (3 x 24) = 299 kips
Deflection = 0.02 x 4.5 x 12 = 1.08"

K = 299 / 1.08 = 277 k/in

B. Structure stiffness can be determined by using the formula for a cantilevered beam. Note that this does not include foundation flexibility.

Assume fixed cantilever with 17.5 foot height.
$K = 3EI / L^3$
$K = 3(3605)(1.99 \times 2 \times 20736) / (17.5 \times 12)^3 = 96$ kips/in

C. The total longitudinal bent stiffness can be determined by summing up the foundation stiffness and the structure stiffness

$K = [(1800 + 277)^{-1} + (316)^{-1} + (96)^{-1}]^{-1} = 71$ k/in

2. Longitudinal abutment stiffness (Page D-7):

Capacity = 400 kips
$K_a = 248$ kips/in

3. Total longitudinal stiffness:

$K_a = 71 + 248 = 319$ kips/in

- Weight:

W = 724 + 653 = 1377 kips

- Natural period:

$$T_{lm} = 2\pi\sqrt{\frac{1377}{386(319)}} = 0.66\,\text{seconds}$$

- Calculate the Longitudinal Seismic Response Coefficient (C_m) for the structure (Page E-2)

$$\text{Long}\;\; C_{lm} = \frac{0.253}{0.66^{2/3}} = 0.33 \le 0.44 \quad \text{Use}\, 0.33$$

- Perform static analysis on the bridge in the longitudinal direction.

1. Calculate the seismic load and deflection.

   P = 0.33(1377) = 454 kips
   δ = 454 / 319 = 1.42 inches < 2" assumed in abutment stiffness (Page D-7)

2. Distribute the seismic load between the abutment and bent based on the relative stiffness.

   Abutment load = 454(248) / 319 = 353 kips < 400 kips (Page D-7) Okay

   Bent load = 454(71) / 319 = 101 kips

3. Determine the seismic loads in the bent columns.

   Longitudinal shear = V = 101 / 2 = 51 kips
   Longitudinal moment = M = 101 x 17.5 / 2 = 884 k-ft

- Combine the loads in each of the two principal directions of the structure to get the final seismic design loads in the bent columns.

  - Transverse column loads:

ITEM	Earthquake		Dead Load	
	LEFT COLUMN	RIGHT COLUMN	LEFT COLUMN	RIGHT COLUMN
AXIAL	-339 k	339 k	371 k	371 k
SHEAR	185 k	185 k	7 k	7 k
M_{+ve}	1391 k-ft	1465 k-ft	35 k-ft	35 k-ft
M_{-ve}	1465 k-ft	1391 k-ft	72 k-ft	72 k-ft

  - Longitudinal column loads:

ITEM	Earthquake	
	LEFT COLUMN	RIGHT COLUMN
AXIAL	-	-
SHEAR	51 k	51 k
M_{top}	-	-
M_{bot}	884 k-ft	884 k-ft

  - Perform two load combinations for investigation:

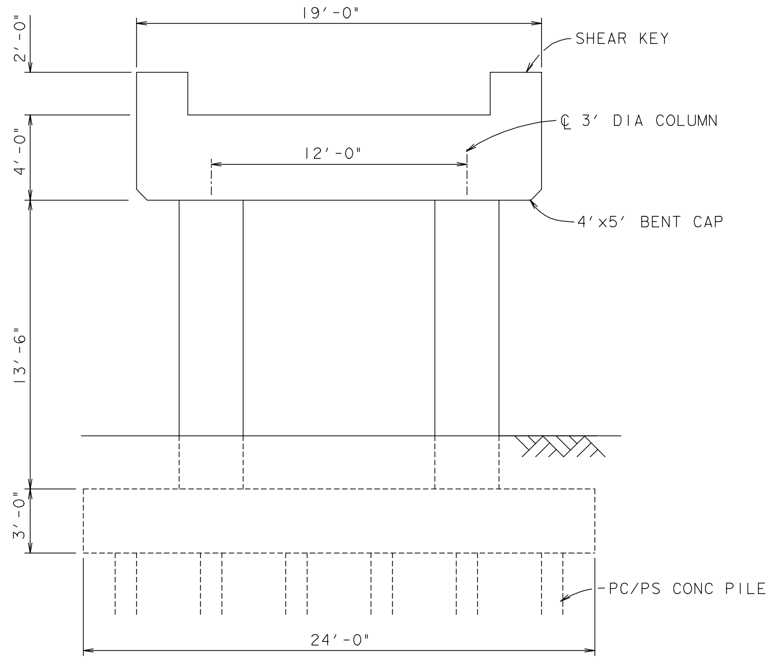
(a) Combination 1: Combine the forces in the longitudinal direction with 30% of the forces from the transverse direction.

$P = 371 +/- 0.3 \times 339 = +473/+269$ kips
$V = [(7 + 0.3 \times 185)^2 + (51)^2]^{1/2} = 81$ kips
$M = [(35 + 0.3 \times 1465)^2 + (884)^2]^{1/2} = 1003$ k-ft

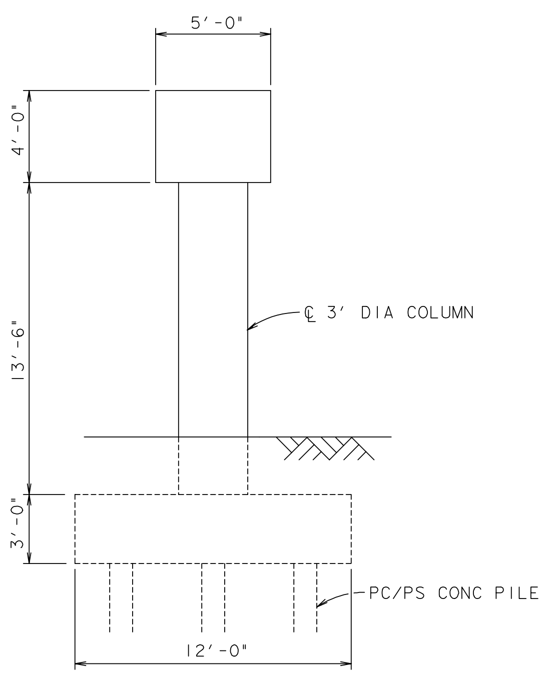
(b) Combination 2: Combine the forces in the transverse direction with 30% of the forces from the longitudinal direction.

$P = 371 +/- 339 = +710/+32$ kips
$V = [(7 + 185)^2 + (0.3 \times 51)^2]^{1/2} = 193$ kips
$M = [(35 + 1465)^2 + (0.3 \times 884)^2]^{1/2} = 1523$ k-ft

BENT DESIGN



BENT ELEVATION - LONGITUDINAL



BENT ELEVATION - TRANSVERSE

Bent loads

- Longitudinal load (AREMA 8-2.2.3j)

  The longitudinal train loads due to braking and adhesion will be calculated.

  (1) Braking LF = 45 + 1.2L = 45 + 1.2(140) = 213 kips
  (2) Adhesion LF = $25(L)^{1/2} = 25(140)^{1/2}$ = 296 kips

  (2) Adhesion LF controls

  AREMA 8-2.2.3j allows for the passive resistance of the backfill behind the abutments to be utilized where applicable. The longitudinal train force to the abutment and bent will be distributed similar to the longitudinal earthquake force on Page E-13.

  Bent load = 296 x 71 / 319 = 66 kips
  Abutment load = 296 x 248 / 319 = 230 kips

  \- Bent column loads:

  V = 66 / 2 = 33 kips
  M_l = 66 x 19 / 2 = 627 k-ft

  The bent column loads are not critical compared to the seismic loads.

- Wind load (AREMA 8-2.2.3h)

  The wind load on structure is calculated using a uniform pressure of 45 psf on the vertical projection of the bridge.

  W = 0.045 x 7.5 x 70 = 24 kips

  The wind loads are not critical compared to the seismic loads.

- Wind on live load (AREMA 8-2.2.3i)

  The wind load on the train is calculated using a linear force of 300 lbs/ft.

  WL = 0.3 x 70 = 21 kips

  The wind on live load is not critical compared to the seismic loads.

Column design

- Load combinations (AREMA 8-2.2.4 & 9-1.4.5)

  Group I: 1.4[D + 5/3(L + I)]
  Seismic: 1.0D + 1.0EQ

  D, L and I loads are shown on page E-8.
  EQ loads are shown on page E-14.

  Group I:
  P = 1.4[371 + 1.67(342 + 89)] = 1527 kips
  V = 1.4[7 + 1.67(7 + 2)] = 31 kips
  M = 1.4[72 + 1.67(73 + 19)] = 316 k-ft

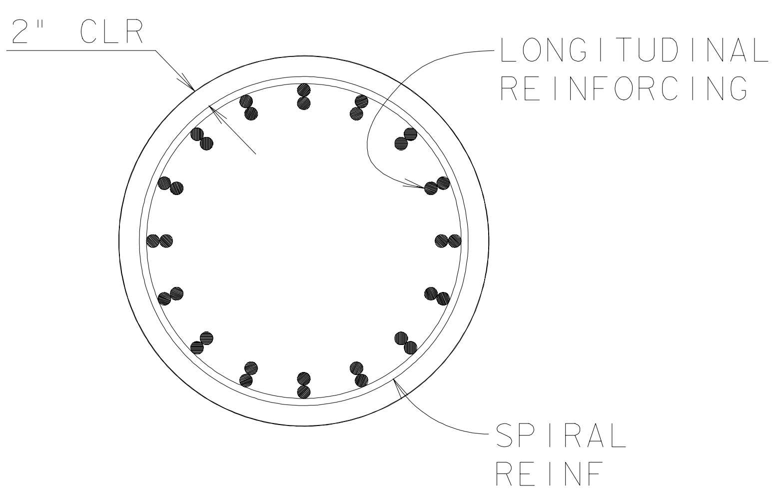
  Seismic-1:
  P = 371 +/- 0.3 x 339 = +473/+269 kips
  $V = [(7 + 0.3 \times 185)^2 + (51)^2]^{1/2} = 81$ kips
  $M = [(35 + 0.3 \times 1465)^2 + (884)^2]^{1/2} = 1003$ k-ft

  Seismic-2:
  P = 371 +/- 339 = +710/+32 kips
  $V = [(7 + 185)^2 + (0.3 \times 51)^2]^{1/2} = 193$ kips
  $M = [(35 + 1465)^2 + (0.3 \times 884)^2]^{1/2} = 1523$ k-ft

- Longitudinal reinforcing design

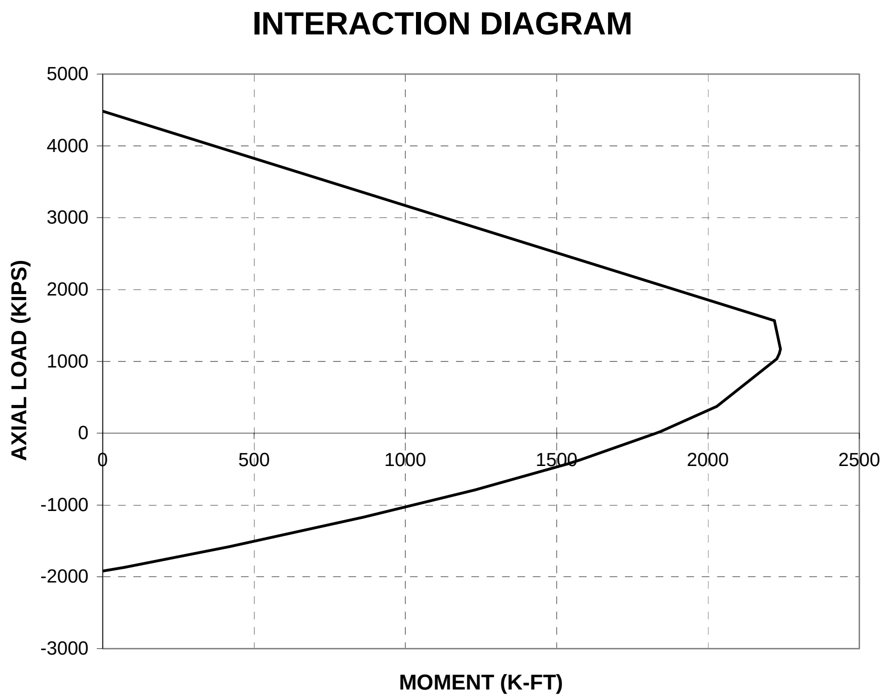
  Try using 32 #9 bars (16-2 bar bundles)

  $A_s = 32\ in^2$
  $\rho_g = 0.031$



COLUMN SECTION

Check compression member with flexure for earthquake load combinations per AREMA 8-2.33. The column analysis is performed using the CONSEC program developed by the author (File = col_36int.out).



c	Mn	Pn	Reinf Tension	Comments
	0	-1920		Maximum tension
1.8	72.3	-1869.2	-1920	
3.6	417.8	-1579.4	-1721	
5.4	862.1	-1168.6	-1493	
7.2	1230.2	-789.4	-1352	
9	1536.8	-426.8	-1232	
9.23	1573.2	-377.4	-1215	
9.26	1577.9	-370.9	-1212	
11.13	1829.1	1.4	-1095	Pure bending
11.3	1851.8	36.1	-1084	
13.02	2029.4	372.5	-980	
16.32	2224.2	1015.5	-782	
16.82	2235.8	1115.4	-750	
16.85	2236.2	1121.6	-747	
17.09	2239.6	1170.5	-731	
19	2219.3	1566.6	-587	Balanced strain
	0	4481.2		Maximum compression

Seismic-2a Combination:

For Axial load = 710 kips
Reduction factor = 0.7
Moment capacity = 1557 k-ft > 1523 Okay

Seismic-2b Combination:

For Axial load = 32 kips
Reduction factor = 0.884
Moment capacity = 1639 k-ft > 1523 Okay

- Longitudinal reinforcing confinement (AREMA 9-1.4.7.2.1)

Design spiral reinforcing to allow column to respond in the post-yield range.

1. AREMA 9-1.4.7.2.1a(1) - The volumetric ratio of spiral reinforcement in the plastic hinge zone shall not be less than:

(1) $\rho_s > 0.12 f'c / fy = 0.12 \times 4 / 60 = .008$
(2) $\rho_s \geq$ that required by Chapter 8, Section 2.11.2

(1) controls by inspection

Volumetric ratio = $\rho_s = 4 \times As / (D \times s)$
Assume #5 spiral: $0.008 < 4 \times 0.31 / (32 \times s)$
$s < 4.84"$

2. AREMA 8-2.11.2a(3) - The longitudinal spacing of the confinement reinforcement in the plastic hinge zone shall not be greater than:

(1) $s \leq$ that required by Chapter 8, Section 2.11.2
(2) $s \leq$ one-quarter of the minimum member dimension
(3) $s \leq$ six times the diameter of the longitudinal reinforcement
(4) $s \leq 6"$ (150 mm)

(1) controls for maximum clear spacing between spirals

Clear s = 3.5 - .625 = 2.875" < 3" Therefore 3.5" spacing is okay

Use #5 spiral at 3.5" spacing throughout column.

Column plastic hinging

The loads due to column plastic hinging in the transverse direction can be calculated either using nonlinear static (pushover) analysis or iterative hand analysis using the following assumptions:

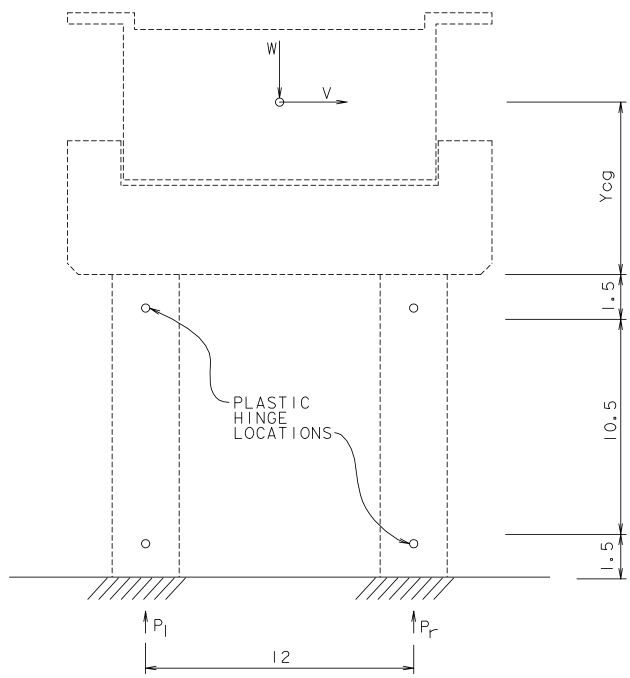
- Column plastic hinging moment per AREMA 9-1.4.7.3.1b
- Column plastic hinge length per AREMA 9-1.4.7.2.1a(5)

The results of this analysis will provide the transverse earthquake design shear force for the columns per AREMA 9-1.4.7.2.1a(7) and the design forces for the bent cap and foundation per AREMA 9-1.4.7.3.1b.

- The length of the plastic hinge zone from the joint face shall not be less than: (AREMA 9-1.4.7.2.1a(5))

  (1) $l_o \geq$ the depth of the member = 36"
  (2) $l_o \geq$ one-sixth of the clear span of the member = 13.5 x 12 / 6 = 27"
  (3) $l_o \geq$ 18" (450 mm)

  (1) 36" controls plastic hinge length



BENT ELEVATION

- Use iterative hand analysis method

Total weight: $W = 371 \times 2 = 742$ kips (Page E-8)
Center of load: $Ycg = 5.5$ ft (Page E-4)

Initial axial loads: $P_l = 371$ kips (Page E-8)
$P_r = 371$ kips

Initial nominal moments: $M_l = 2029$ k-ft (Page F-4)
$M_r = 2029$ k-ft

Total lateral load: $V = 2 \times 1.3\ (2029 + 2029) / 13.5 = 782$ kips

Iteration 1: $P_r = [742 \times 6 + 782 \times 19 - 1.3(2029+2029)]/12 = 1169$ kips
$P_l = 742 - 1169 = -427$ kips
$M_l = 1537$ k-ft (page F-4)
$M_r = 2240$ k-ft
$V = 2 \times 1.3\ (1537 + 2240) / 13.5 = 727$ kips

Continue iterations until V converges:

No	P_l	P_r	M_l	M_r	V
1	-427	1169	1537	2240	727
2	-372	1114	1578	2236	735
3	-379	1121	1573	2236	734

Use iteration number 3 for final loads (without 1.3 factor on moments).

- Compare unreduced level 3 loads with plastic hinging loads

The unreduced level 3 loads can be calculated by scaling the level 1 loads by the ratio of the base acceleration coefficients.

Factor = 0.59 / 0.22 = 2.68 (Page B-4)

Axial load: $P_{min} = 371 - 2.68 \times 339 = -538$ kips
$P_{max} = 371 + 2.\ 68 \times 339 = 1280$ kips

Shear: $V = 7 + 185 \times 2.68 = 503$ kips (Page E-14)

Moment: $M_{rtop} = 72 + 1391 \times 2.68 = 3800$ k-ft
$M_{rbot} = 35 + 1465 \times 2.68 = 3961$ k-ft
$M_{ltop} = 72 - 1391 \times 2.68 = -3656$ k-ft
$M_{lbot} = 35 - 1465 \times 2.68 = -3891$ k-ft

- Check column shear

1. AREMA 9-1.4.7.2.1a(7) - The column shear is based on the maximum force which can be generated.

   $V_u = [(7 + 185 \times 2.68)^2 + (0.3 \times 51 \times 2.68)^2]^{1/2} = 504$ kips (Level 3)

   $V_{1u} = 1.3 \times 2 \times 1573 / 13.5 = 303$ kips (Column 1 plastic hinging)
   $V_{2u} = 1.3 \times 2 \times 2236 / 13.5 = 431$ kips (Column 2 plastic hinging)

   The plastic hinging shear load controls since it is less than the unreduced level 3 shear load.

2. AREMA 9-1.4.7.2.1a(8) - The confinement reinforcement in the plastic hinge zone shall be proportioned to resist shear assuming the nominal concrete shear strength is zero when the shear force determined in Paragraph 1.4.7.2.1a(7) is greater than one-half the maximum required shear strength in this area and the factored axial compressive force for the seismic load condition is less than $A_g f'_c/20$.

   Maximum required shear strength = 193 kips (Page F-3)
   303 kips and 431 kips > 193 / 2 = 97 kips

   -and -

   Axial compressive force Pu (min) = -379 kips < $A_g f'_c/20 = 204$ kips
   Axial compressive force Pu (max) = 1121 kips > 204 kips

   Therefore the concrete shear strength, ϕV_c , is assumed to be zero for the column in tension.

3. AREMA 8-2.35.2 - Concrete shear strength

   For circular column: $\phi V_n = \phi \times 2 (f'_c)^{1/2} \times 0.8A_c$
   $\phi V_n = 0.85 \times 2 (4000)^{1/2} \times 0.8 \times 1018 / 1000 = 88$ kips

   Shear reinforcing strength $\phi V_s > 431 - 88 = 343$ kips

4. AREMA 8-2.35.3 - Design of shear reinforcement

For spiral reinforcing: $\phi V_s = \phi\ A_v\ f_y\ D'\ \pi / (2 \times s)$

Check #5 spiral @ 3.5"
D' = 36 - 4 - 0.625 = 31.4"
s = 3.5"
ϕV_s = 0.85 x 0.31 x 60 x 31.4 x 3.1416 / (2 x 3.5) = 222 kips < 343 NG

Increase confinement reinforcement:

Try #6 spiral @ 3"
ϕV_s = 0.85 x 0.44 x 60 x 31.4 x 3.1416 / (2 x 3) = 369 kips > 343 OK

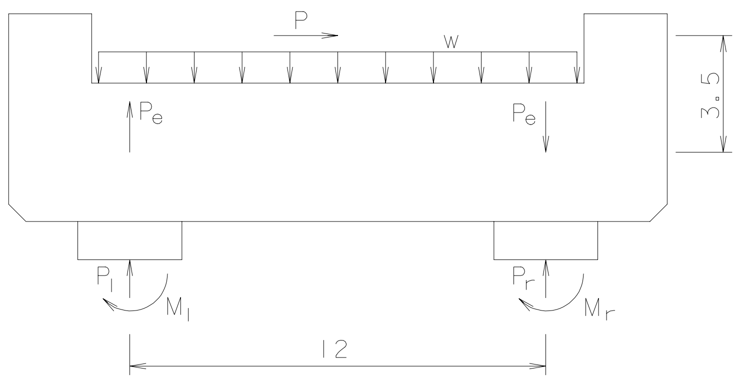
Use #6 spiral at 3" pitch

Bent cap design

- Load combinations (AREMA 8-2.2.4 & 9-1.4.5)

  Group I: 1.4[D + 5/3(L + I)]
  Seismic: 1.0D + 1.0EQ

  D+L+I loads are shown on page E-8.
  EQ loads are shown on page F-7.



BENT CAP FREE BODY DIAGRAM

D+L+I : w = 1.4[49.64 + 1.67(48.86 + 12.71)] = 213 k/ft
V = 1.4[298 + 1.67(293 + 76)] - 213 x 2 = 854 kips
M_{+ve} = 1.4[780 + 1.67(782 + 203)] = 3395 k-ft

EQ: w = 49.64 k/ft
P_e = 734 x 5.5 / 12 = 336 kips (lateral load above cap cl)
$M_{+ve} = 2045 - 49.64(2.5)^2 / 2 - (379 - 336) \times 1.5 = 1825$ k-ft
$M_{-ve} = 2907 + 49.64(2.5)^2 / 2 - (1121 - 336) \times 1.5 = 1885$ k-ft

- Flexural design per AREMA 8-2.32

Positive moment

Mu_{+ve} = 1825 k-ft
b = 60", h = 48", d = 44"
$As_{+ve} > 9.52$ in^2

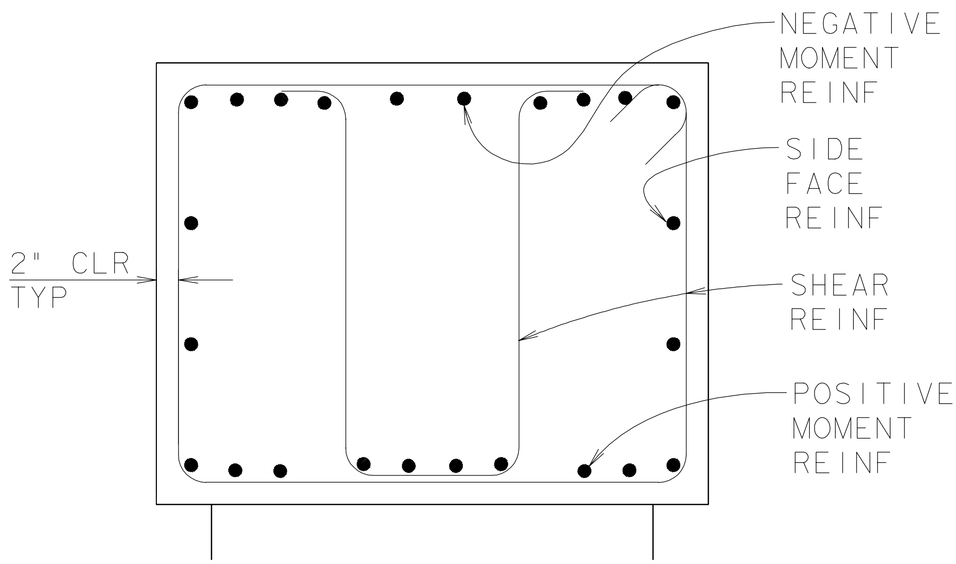
AREMA 9-1.4.7.3.2b(3)
Add 8% column steel
0.08 x 32 = 2.56 in^2
Total As_{+ve} = 12.08 in^2

Negative moment

Mu_{-ve} = 1885 k-ft
b = 60", h = 48", d = 44"
$As_{-ve} > 9.84$ in^2

AREMA 9-1.4.7.3.2b(3)
Add 8% column steel
0.08 x 32 = 2.56 in^2
Total As_{-ve} = 12.4 in^2

Try (10) #10 top and bottom, As = 12.7 in^2 > 12.4 Okay



BENT CAP SECTION

- Shear design per AREMA 8-2.35

V_u = 854 kips
b = 60", h = 48", d = 44"

$\phi V_c = 0.85 \times 2\,(4000)^{1/2} \times 60 \times 44 / 1000 = 284$ kips

$\phi V_s > 854 - 284 = 570$ kips
Assume 4 legs of #6 stirrups
$s < 0.85 \times 4 \times .44 \times 60 \times 44 / 570 = 6.92$" use 6"

This spacing may be increased as the shear is reduced away from the joint.

Column / Cap Joint Design

- Joint shear design per AREMA 9-1.4.7.3.2b

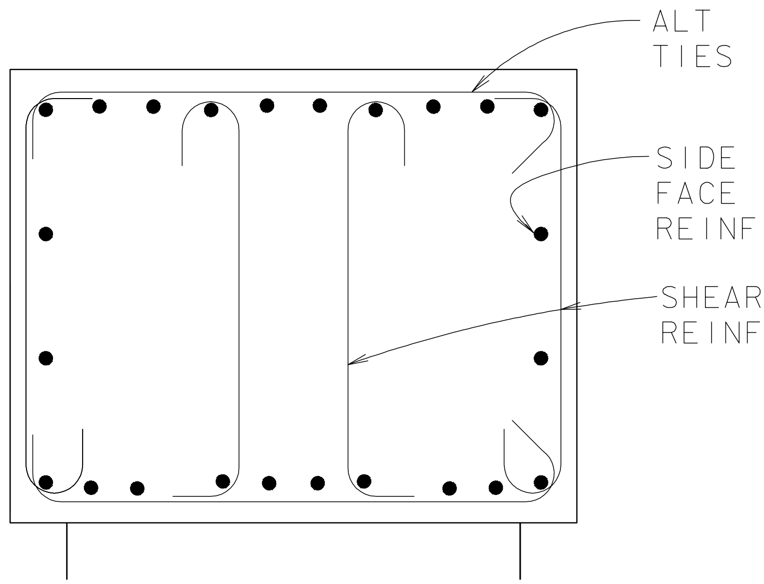
  Outside the joint (within a distance of 1/2 the column width):

  $A_v > 0.16 \times 32 = 5.12 \text{ in}^2$
  no. of 4-legged #6 stirrups = 5.12 / (0.44 x 4) = 2.9 say 3
  s < 18 / (3 - 1) = 9" > 6" therefore the regular shear design controls at the joint

  Inside the joint:

  $A_v > 0.08 \times 32 = 2.56 \text{ in}^2$
  no. of 4-legged #6 stirrups = 2.56 / (0.44 x 4) = 1.5 say 2

  Use 2 stirrups inside the joint - revise the closed stirrup detail as shown below for constructability.



BENT CAP SECTION AT COLUMN

- Column reinforcing development (AREMA 9-1.4.7.3.2a(1))

  Column longitudinal reinforcement shall extend as close as practical to the far face of the adjoining member, but not less than:

  For hooked bars in tension:

  $l_{dh} \geq$ that required by Chapter 8, Section 2.17

  $l_{dh} \geq 8d_b$

  $l_{dh} \geq 6"$

  $$l_{dh} \geq \frac{f_y d_b}{65\sqrt{f_c^{'}}} \text{ inches}$$

  For straight bars:

  $l_d \geq$ that required by Chapter 8, Sections 2.14 through 2.16

  $l_d \geq$ 2.5 times that required in this Article for hooked bars in tension

  Straight bars must be used since there is no room to hook the #9 bundled bars. The preferred available development length = 48 - 2 - 0.875 - 1.438 - 2 = 41.7" to avoid the top main cap reinforcing.

  (1) $l_d = 2.5 \times 60000 \times 1.128 / (65 \times (4000)^{1/2}) = 41"$
  (2) $l_d = 0.04 \times 1.0 \times 60000 \times 0.75 / (4000)^{1/2} = 28"$ (AREMA 8-2.14)

  (1) controls, therefore a 41" development length will be used for the #9 column reinforcing.

- Column confinement reinforcement (AREMA 9-1.4.7.3.2a(2))

  Confinement reinforcement shall be provided throughout the joint to the end of the longitudinal column reinforcement in an amount equal to the greater of that specified in Article 9-1.4.7.2.1a or Part b of Article 9-1.4.7.3.2.

  - AREMA 9-1.4.7.2.1a -

    required #5 @ 4.84" (Page F-5)

  - AREMA 9-1.4.7.3.2b(4) -

    $\rho_s > 0.4 \times A_s / (l_d)^2$
    $\rho_s = 0.4 \times 32 / (41)^2 = 0.0076$

    Volumetric ratio = $\rho_s = 4 \times A_s / (D \times s)$
    Assume #5 spiral: $0.0076 < 4 \times 0.31 / (32 \times s)$
    $s < 5.1"$

  Use #5 welded hoops at 4" pitch to confine joint.

- Maximum joint shear (AREMA 9-1.4.7.3.2a(3))

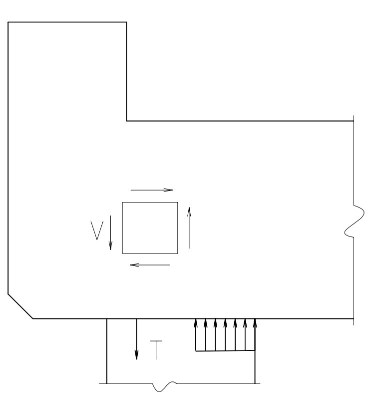
  The nominal shear strength of the joint shall not be taken greater than:

$$v_c = 20\sqrt{f_c'} \text{ psi}$$

$$\phi V_c = \phi 20\sqrt{f_c'}bd$$

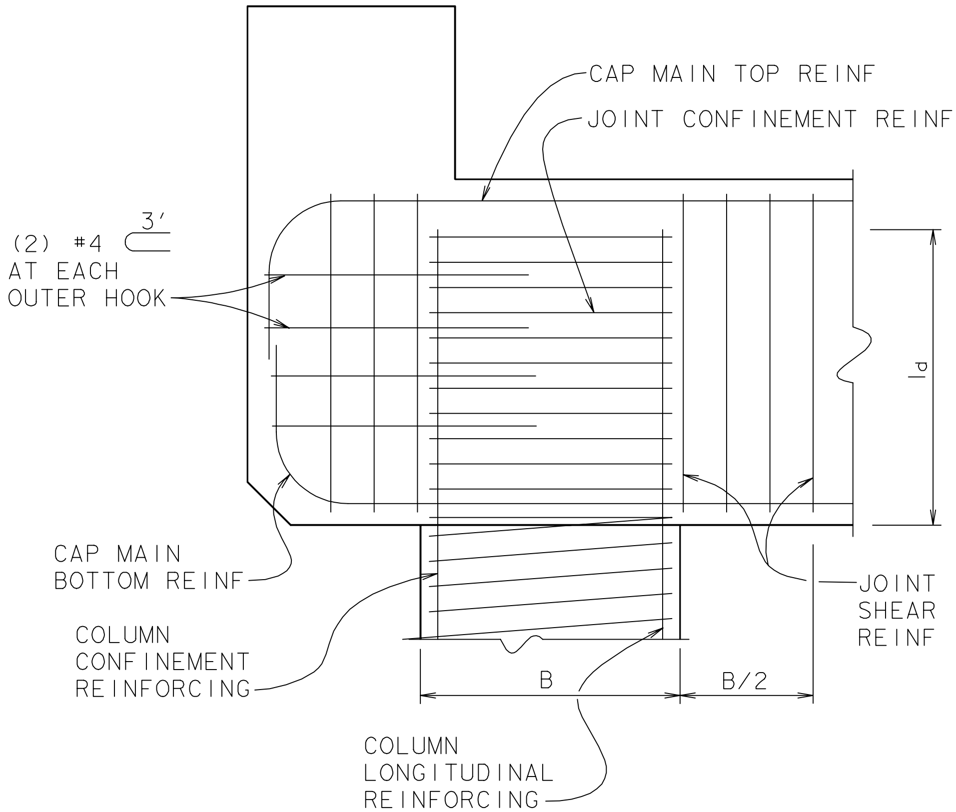
$$\phi V_c = 0.85 \times 20\,(4000)^{1/2} \times 60 \times 44 / 1000 = 2838 \text{ kips}$$

The maximum joint shear can be approximated using the reinforcing tension from the interaction diagram shown on Page F-4 for the pure bending case.



T = 1095 kips
$V_u = 1095 \times 1.3 = 1424$ kips < 2838 kips Okay

- Column / Cap Joint Detail



1. Hook cap main reinforcement beyond joint and restrain hooks with #4 hairpins as shown.

2. Provide joint shear reinforcement of #6 @ 6" with a closed outer hoop as shown on sheet F-11 outside of the joint.

3. Provide joint shear reinforcement of (2) #6 with modified stirrups as shown on sheet F-12 inside of joint.

4. Develop column longitudinal reinforcing with l_d = 41" into the cap.

5. Provide column confinement reinforcing of #5 hoops @ 4" inside the joint.

6. Provide column confinement reinforcing of #6 spirals @ 3" outside the joint (discontinue at cap main bottom reinforcing).

Foundation Design

- Pile loads (AREMA 8-2.2.4 & 9-1.4.5)

Weight of pile cap and soil:

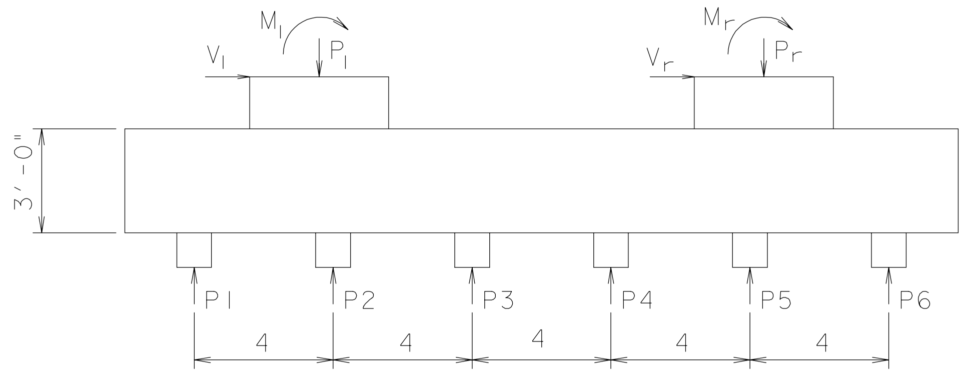
$W = 24 \times 12 \times 3 \times 0.15 + (24 \times 12 - 7.07 \times 2)(1.5 \times 0.12) = 179$ kips

- Service loads: D + L

D+L loads are shown on page E-8.

$P_l = P_r = 371 + 342 = 713$ kips
Column shears and moment cancel each other out.

$P1 = (713 \times 2 + 179) / 12 = 134$ kips



PILE CAP TRANSVERSE FREE BODY DIAGRAM

- Ultimate Group I: 1.4[D + 5/3(L)]

$P_l = 1.4[371 + 1.67(342)] = 1319$ kips $P_r = 1319$ kips
$V_l = -1.4[7 + 1.67(7)] = -26$ kips $V_r = 26$ kips
$M_l = -1.4[35 + 1.67(35)] = -131$ k-ft $M_r = 131$ k-ft

$P1 = (1319 \times 2 + 1.4 \times 179) / 12 = 241$ kips

- Seismic: 1.0D + 1.0EQ

1. Transverse EQ load combination is shown on page F-7 and F-8 for the column plastic hinging load.

   P_l = -379 kips　　P_r = 1121 kips
   V_l = 303 kips　　V_r = 431 kips
   M_l = 1.3 x 1573 = 2045 k-ft　　M_r = 1.3 x 2236 = 2907 k-ft

   Resultant loads:

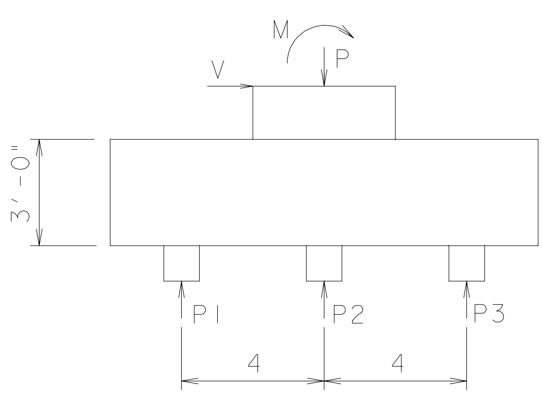
   P = 1121 - 379 + 179 = 921 kips
   V = 303 + 431 = 734 kips, V_p = 734 / 18 = 41 kips/pile
   M = 2045 + 2907 + 734 x 3 = 7154 k-ft

   $I_{piles} = 6(2)^2 + 6(6)^2 + 6(10)^2 = 840$

   P1 = 921 / 18 - 7154 x 10 / 840 = -34 kips
   P6 = 921 / 18 + 7154 x 10 / 840 = 136 kips

2. Longitudinal EQ loads are controlled by the column plastic hinging loads. These loads are obtained from the interaction diagram shown on page F-4 using the 371 kip column dead load.

   P = 371 x 2 + 179 = 921 kips
   1.3 x M_n = 1.3 x 2029 = 2638 k-ft
   V = 2638 x 2 / (5.5 + 13.5) = 278 kips
   M = 2638 x 2 + 278 x 3 = 6110 k-ft



PILE CAP LONGITUDINAL FREE BODY DIAGRAM

P1 = 921 / 18 - 6110 / (6 x 8) = -76 kips
P3 = 921/ 18 + 6110 / (6 x 8) = 178 kips

- Final pile design loads

Service:

P = 134 kips

Seismic:

P = 178 kips
P = -76 kips (tension)

Design pile lengths for 70 ton capacity under service loads and100 ton capacity under seismic loads.

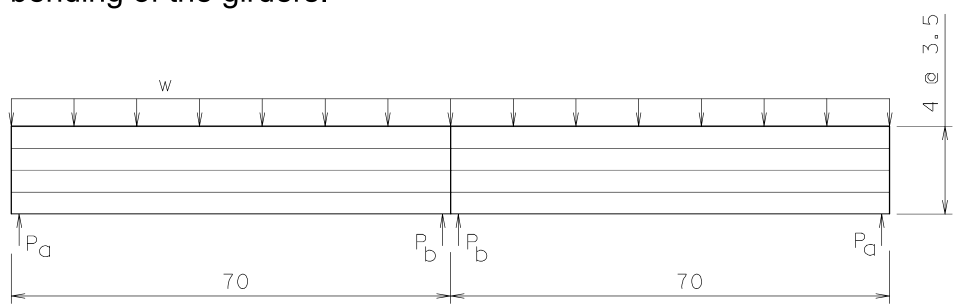
- Additional foundation design calculations are required to determine the footing reinforcement, however, they are not included in this example.

CONTINUITY PROVISIONS

The structure shall be designed with an uninterrupted load path to transfer lateral forces from the superstructure to the ground (AREMA 9-1.4.7.1).

Superstructure continuity (AREMA 9-1.4.7.1.1)

The superstructure shall be designed to carry the lateral forces to the bearings or shear connectors. The lateral forces from the span will be carried to the end supports by lateral bending of the girders.



PLAN - LATERAL BENDING OF GIRDERS

- Lateral bending of girders (Level 1)

  w = 4.54 k/ft (Page E-9)
  V = 4.54 x 70 / 2 =159 kips
  M = 4.54 x $(70)^2$ / 8 = 2781 k-ft

  Prestressing load (Page C-3):

  P = 947 x 4 = 3788 kips
  e = 26.08"

  Dead load (Page C-1):

  V = 189 + 111 = 300 kips (4 girders)
  M = 3189 + 1879 = 5068 k-ft (4 girders)

  Section properties (Page C-3):

  A = 1316 x 4 = 5264 in^2
  I_x = 678151 x 4 = 2713000 in^4
  I_y = 312359 x 4 + 1316 x 2 x 2 $(42)^2$ = 10535000 in^4

Bending stress:

- Conservatively assume that the girders act separately

$\sigma_{(DL+PS)}$ = 947 / 5264 - (5068 x 12 - 3788 x 26.08) x 33 / 2713000 = 0.642 ksi
σ_{EQ} = 2781 x 12 x 21 / (4 x 312359) = 0.561 ksi < 0.642 ksi Okay
(Girders still in compression)

Note: The box girders are not considered a non-ductile, non-redundant primary load carrying member per AREMA 9-1.4.4.1.2 for lateral bending, therefore there is no need to check the Level 3 load transfer.

Bearings (AREMA 9-1.4.7.1.2)

The elastomeric bearings cannot be relied upon to transfer lateral earthquake forces since their lateral load resistance depends on the amount of friction developed. Shear connectors and span ties will be designed to transfer the lateral seismic loads at the abutments and bent.

- Shear connectors (AREMA 9-1.4.7.4.2)

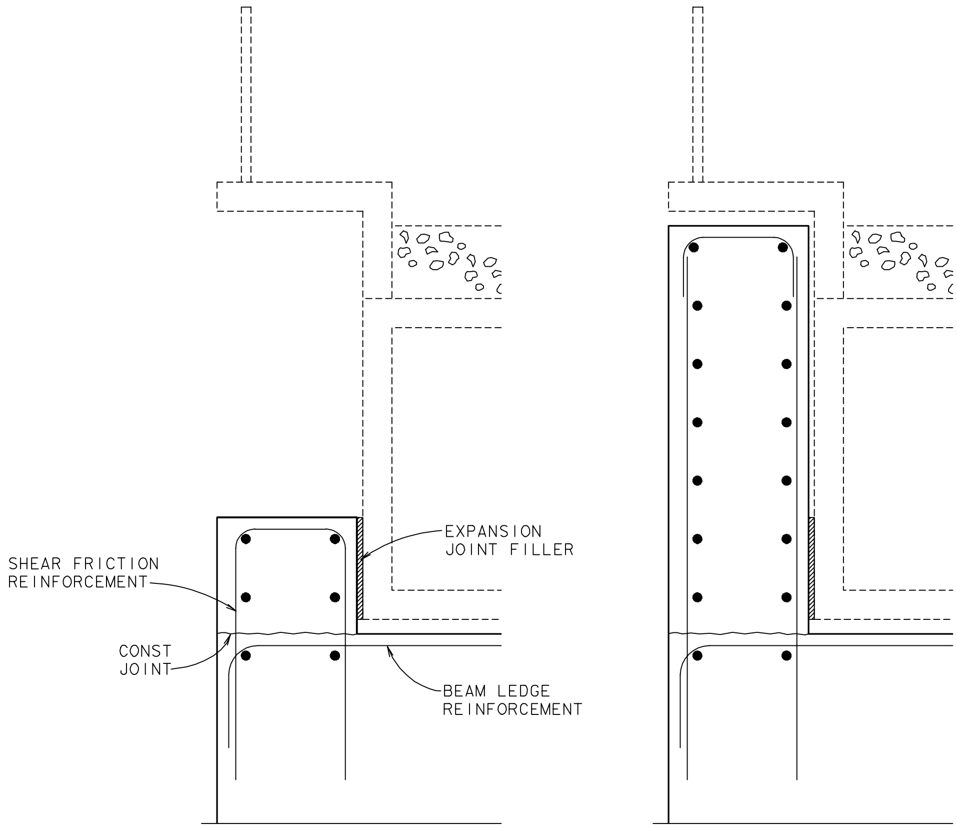
  Shear connectors consisting of reinforced concrete shear keys will be provided to transfer the transverse seismic loads to the abutments and bent.

  - Dimensions:

    Shear key width = 36 - 7 - 1 = 28" (Page C-1)
    Shear key height = 24" at bent (Page F-1)
    Shear key length = 30" per girder support
    Area = 30 x 28 = 840 in^2

  - Transverse seismic load (Level 1):

    V = 180 kips (Page E-9)
    M = 180 x 2 /2 = 180 k-ft



BENT ABUTMENT

SHEAR KEYS

- Shear friction reinforcement design (AREMA 8-2.35.4):

Maximum shear resistance = 0.85 x 0.8 x 840 = 571 kips > 180 kips Okay
A_{vf} > 180 / (0.85 x 60 x 1.0) = 3.53 in^2

Flexural reinforcement:

b = 30", d = 28 - 2.5 = 25.5", A_s > 1.65 in^2

At abutments, use (6) #5 A_s = 1.86 > 1.65 Okay
A_{vf} = 3.72 > 3.53 Okay
At bents, use (12) #5

Note: The transverse shear keys are not considered a non-redundant primary load carrying member per AREMA 9-1.4.4.1.2, therefore there is no need to check the Level 3 load transfer.

- Beam ledge reinforcement design

Note: To limit damage during a higher level earthquake, the beam ledge will be designed for the maximum force which can be transmitted by the transverse shear keys. The maximum shear key load will be calculated assuming a strength reduction factor of 1.0 and a 1.4 shear friction factor.

$P = 3.72 \times 60 \times 1.4 = 312$ kips

1. Abutment reinforcement

Design is needed for the beam ledge reinforcement at the abutment seat to prevent a shear failure at the abutment corner.

$V = 312 - 0.85 \times 2(4000)^{1/2} \times 30 \times 25.5 / 1000 = 230$ kips

$A_s > 230 / (0.9 \times 60) = 4.26\ in^2$

Use (6) #8 at the abutment seat

$A_s = 4.74\ in^2 > 4.26\ in^2$ Okay

2. Bent cap reinforcement

$V = 2 \times 230 = 460$ kips

$A_s > 460 / (0.9 \times 60) = 8.52\ in^2$

(10) #10 are available at the bent cap:

$A_s = 12.7\ in^2 > 8.52\ in^2$ Okay

- Span ties (AREMA 9-1.4.7.4.3)

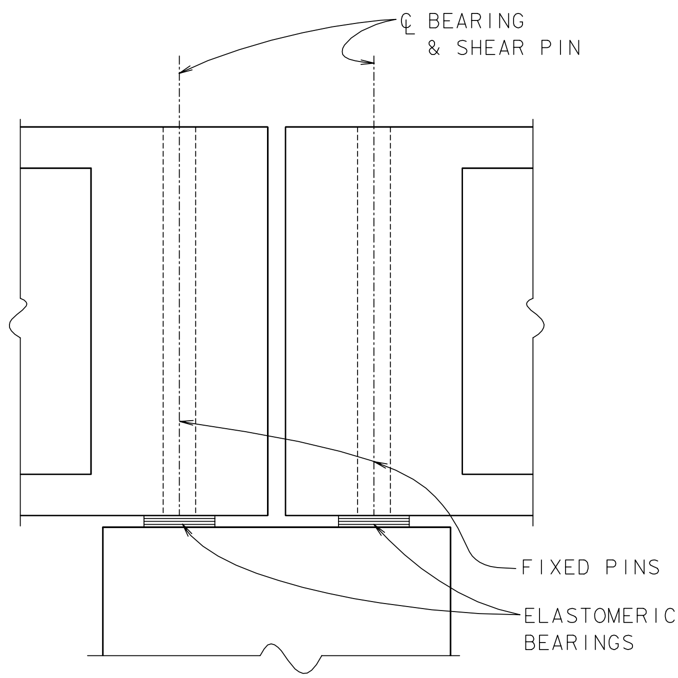
  The spans will be tied together through the bent cap with shear pins to transfer the longitudinal seismic loads to the bent.

  Note: Failure of the longitudinal span ties will probably not result in unseating the spans since wide bearing seats are used. The shear pins will be designed for the Level 3 seismic load since they are difficult to access for repair after a lower level earthquake.

  - Longitudinal seismic load (Level 3):

    V = 101 x 0.59 / 0.22 = 270 kips (Page E-13)

  - Shear pin design



SHEAR PIN CONNECTION

One shear pin per girder will be used to transfer the longitudinal load.

P = 270 / 8 = 33.8 kips

$$P = \frac{\pi d^2 f_y}{4\sqrt{3}}$$

Note that this formula assumes the pin shear strength $f_v = \frac{f_y}{\sqrt{3}}$

Minimum pin diameter > $(33.8 \times 2.2 / 130)^{1/2} = 0.76"$

Try 1.5" diameter A354 grade BD pin, $f_y = 130$ ksi

Check pin bending through the depth of the elastomeric bearing pad.

Pad thickness = 2.9" (Page C-11)
M = PL / 2 = 33.8 x 2.9 / 2 = 49 k-in
$Z = (1.5)^3 / 6 = 0.56\ in^3$
$f_b = 49 / 0.56 = 88$ ksi < 130 ksi Okay