SEISMIC DESIGN EXAMPLE FOR RAILROAD UNDERPASS

Prepared by:

Robert Matthews
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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](image-url)
Configure bridge:

Bridge length ≅ 2 x (48 + (16 + 0.02 x 36 - 0.67 - 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)

<table>
<thead>
<tr>
<th>PREFERRED CONFIGURATION</th>
<th>SPECIAL CONSIDERATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>✓ Straight bridge alignment</td>
<td>Curved bridge alignment</td>
</tr>
<tr>
<td>✓ Normal piers</td>
<td>Skewed piers</td>
</tr>
<tr>
<td>✓ Uniform pier stiffness</td>
<td>Varying pier stiffness</td>
</tr>
<tr>
<td>✓ Uniform span stiffness</td>
<td>Varying span stiffness</td>
</tr>
<tr>
<td>✓ Uniform span mass</td>
<td>Varying span mass</td>
</tr>
</tbody>
</table>

- The preferred bridge configuration will be used in all cases

- Superstructure configuration: (AREMA 9-1.4.3.2)

<table>
<thead>
<tr>
<th>PREFERRED SUPERSTRUCTURE</th>
<th>SPECIAL CONSIDERATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>✓ Simple spans</td>
<td>Continuous spans</td>
</tr>
<tr>
<td>✓ Short spans</td>
<td>Long spans</td>
</tr>
<tr>
<td>Light spans</td>
<td>✓ Heavy spans</td>
</tr>
<tr>
<td>✓ No hinges</td>
<td>Intermediate hinges</td>
</tr>
</tbody>
</table>

- (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 \( \approx 0.08 \times (70-2.5) = 5.4 \text{ ft.} \)

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.

```
- Substructure configuration: (AREMA 9-1.4.3.3)

<table>
<thead>
<tr>
<th>PREFERRED SUBSTRUCTURE</th>
<th>SPECIAL CONSIDERATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>✓ Wide seats</td>
<td>Narrow seats</td>
</tr>
<tr>
<td>✓ Seat bent caps</td>
<td>Integral bent caps</td>
</tr>
<tr>
<td>✓ Multiple column</td>
<td>Single column</td>
</tr>
</tbody>
</table>

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

\[
N = (12 + 0.03L + 0.12H)(1 + 0.000125S^2) \text{ inches}
\]

- \( L \) = 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).

\[
N = [12 + 0.03(70) + 0.12(18)](1) = 16.3 \text{ inches}
\]

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 Diagram]

  **BENT ELEVATION**
The conceptual bridge configuration is shown below.
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 = $(1 - \sin 32.5) / (1 + \sin 32.5) \times 120 = 36$ pcf
Loads:

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

![Cooper E80 Load](image)

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

- 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.

\[ n = \ln \left( \frac{0.25}{0.50} \right) \approx -1.558 \]

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

- Return period = 358 years

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

- Return period = 1980 years

\[ A_{1980} = e^x \]

\[ x = \ln \left( A_{475} \right) + \left[ \ln \left( A_{2400} \right) - \ln \left( A_{475} \right) \right] \left[ 0.6061 \ln \left( P_R \right) - 3.73 \right] \]

\[ x = \ln (0.50) + \left[ \ln (0.60) - \ln (0.50) \right] \left[ 0.6061 \ln (1980) - 3.73 \right] = -0.535 \]

\[ A_{1980} = e^{-0.535} \approx 0.59 \]
SUPERSTRUCTURE DESIGN

Box Girder Design:

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

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.

<table>
<thead>
<tr>
<th>SPAN LENGTH (FT)</th>
<th>MAXIMUM MOMENT (KIP-FT)</th>
<th>MAXIMUM MOMENT 1/4 POINT (KIP-FT)</th>
<th>MAXIMUM SHEAR (KIPS)</th>
<th>MAXIMUM PIER REACTION (KIPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AT END</td>
<td>AT 1/4 POINT</td>
<td>AT CENTER</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2597.80</td>
<td>2010.00</td>
<td>196.00</td>
<td>120.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3415.00</td>
<td>2608.20</td>
<td>221.04</td>
<td>131.89</td>
</tr>
</tbody>
</table>

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)

\[ I = 14 + \frac{800}{(67.5 - 2)} = 26\% \]

• 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 \times 8092) = 26014 kip-ft
  Shear = 1.4(300 + 1.67 \times 542) = 1687 kips
  Pier reaction = 1.4(653 + 1.67 \times 862) = 2930 kips
Design Prestressing Steel:

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

- Beam properties:
  
  Area = 1316
  $I_x = 678151$
  $I_y = 312359$
  $Y_{\text{top}} = Y_{\text{bot}} = 33$
  $Y_{\text{side}} = 21$

- Eccentricity:

  \[
  \frac{32(3.5)+4(6.5)+2(62.5)}{38}=6.92'' \\
  e = 33 - 6.92 = 26.08''
  \]

**BOX GIRDER SECTION**

- Stresses (AREMA 8-17.6.4)

  Tension at bottom = $f_t = \frac{13160 \times 12 \times 33}{(678151 \times 4)} = 1.921 \text{ ksi}$
  
  Compression on top = $f_c = -1.921 \text{ ksi}$
  
  Maximum tension allowed = 0 ksi

- Required prestress force after all losses:

  \[
  P = 1.921 / (1 / 1316 + 26.08 \times 33 / 678151) = 947 \text{ kips}
  \]

  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.
**SEISMIC DESIGN EXAMPLE FOR RAILROAD UNDERPASS**

**PROGRAM OPTIONS**

<table>
<thead>
<tr>
<th>Units</th>
<th>English (inches, pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Criteria</td>
<td>AREA</td>
</tr>
<tr>
<td>Prestressing</td>
<td>Pretensioning</td>
</tr>
<tr>
<td>Section Properties</td>
<td>Gross</td>
</tr>
</tbody>
</table>

**MATERIAL PROPERTIES**

**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

**SECTION PROPERTIES**

**BEAM SECTION AND DIMENSIONS**

<table>
<thead>
<tr>
<th>Box Girder</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>66.000</td>
</tr>
<tr>
<td>Btop</td>
<td>42.000</td>
</tr>
<tr>
<td>Htop</td>
<td>7.000</td>
</tr>
<tr>
<td>Bweb</td>
<td>7.000</td>
</tr>
<tr>
<td>Bbot</td>
<td>42.000</td>
</tr>
<tr>
<td>Hbot</td>
<td>7.000</td>
</tr>
<tr>
<td>Ftop</td>
<td>0.000</td>
</tr>
<tr>
<td>Fbot</td>
<td>0.000</td>
</tr>
</tbody>
</table>
BEAM PROPERTIES

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

LOADS

Initial prestress force = 1177335.0
Number of load locations = 1

LOCATION NUMBER 1 AT X = 405

<table>
<thead>
<tr>
<th>CASE</th>
<th>AXIAL</th>
<th>SHEAR</th>
<th>MOMENT</th>
<th>FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEAM DEAD</td>
<td>0.0</td>
<td>0.0</td>
<td>9567000.</td>
<td>1.400</td>
</tr>
<tr>
<td>NON-COMP DEAD</td>
<td>0.0</td>
<td>0.0</td>
<td>5637000.</td>
<td>1.400</td>
</tr>
<tr>
<td>COMPOSITE DEAD</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.000</td>
</tr>
<tr>
<td>LIVE PLUS IMPACT</td>
<td>0.0</td>
<td>0.0</td>
<td>24276000.</td>
<td>2.330</td>
</tr>
<tr>
<td>PRESTRESS</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.000</td>
</tr>
<tr>
<td>SECONDARY PRESTRESS</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.000</td>
</tr>
<tr>
<td>MISCELLANEOUS</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.000</td>
</tr>
</tbody>
</table>

ALLOWABLE STRESS AND LOSS

ALLOWABLE STRESS

Initial concrete compressive stress = 2400.
Initial concrete tensile stress     = 474.
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

CONFIGURATION

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
**PRESTRESS FORCES**

* * * * * * * * * * * * * * * * * * * * * * * * * * *
* PROGR A M  PSBEAM *
* OUTPUT DATA *
* * * * * * * * * * * * * * * * * * * * * * * * * * *

LOCATION NUMBER 1 AT X = 405

<table>
<thead>
<tr>
<th>CASE</th>
<th>VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL FORCE</td>
<td>1177335.</td>
</tr>
<tr>
<td>CABLE ECCENTRICITY</td>
<td>-26.08</td>
</tr>
<tr>
<td>FORCE AT TRANSFER</td>
<td>1101876.</td>
</tr>
<tr>
<td>MOMENT AT TRANSFER</td>
<td>-28736912.</td>
</tr>
<tr>
<td>FINAL FORCE</td>
<td>953939.</td>
</tr>
<tr>
<td>FINAL MOMENT</td>
<td>-24878738.</td>
</tr>
</tbody>
</table>

**BEAM STRESSES**

* * * * * * * * * * * * * * * * * * * * * * * * * * *

LOCATION NUMBER 1 AT X = 405

Prestress loss = 38424

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

CHECK CRITICAL CONCRETE STRESSES

<table>
<thead>
<tr>
<th>CASE</th>
<th>ACTUAL</th>
<th>ALLOWABLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>INITIAL TENSION</td>
<td>-96.</td>
<td>-474.</td>
</tr>
<tr>
<td>INITIAL COMPRESSION</td>
<td>1770.</td>
<td>2400.</td>
</tr>
<tr>
<td>FINAL TENSION (DL ONLY)</td>
<td>1196.</td>
<td>0.</td>
</tr>
<tr>
<td>FINAL TENSION</td>
<td>14.</td>
<td>0.</td>
</tr>
<tr>
<td>FINAL COMPRESSION</td>
<td>1435.</td>
<td>2000.</td>
</tr>
</tbody>
</table>
### Moment Capacity

<table>
<thead>
<tr>
<th>X</th>
<th>Mult</th>
<th>1.2Mcr</th>
<th>phiMn</th>
</tr>
</thead>
<tbody>
<tr>
<td>405</td>
<td>77848680.</td>
<td>13077953.</td>
<td>76452936.</td>
</tr>
</tbody>
</table>

### Beam Deflections

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

### Shear Capacity

<table>
<thead>
<tr>
<th>X</th>
<th>Vult</th>
<th>phiVn</th>
<th>Av/S</th>
</tr>
</thead>
<tbody>
<tr>
<td>405</td>
<td>0.</td>
<td>89484.</td>
<td>0.012</td>
</tr>
</tbody>
</table>
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 = \( A > \frac{(39.4 + 67.7)}{1.0} = 107 \text{ in}^2 \)
    Square bearing width > \( (107)^{1/2} = 10.3" \)

    Try 12" x 12" bearing (\( A = 144 \text{ in}^2 \))

    Compressive stress = \( f_a = \frac{107.1}{144} = 0.74 \text{ ksi} < \frac{G S}{k} \)
    Shear modulus = \( G = 140 \text{ psi} \) (Figure 1-1c at 70 degrees F)
    Shape factor = \( S = \frac{L \times W}{2 \times t_i (L + W)} \)
    \( S = \frac{12 \times 12}{2 \times 0.5 (12 + 12)} = 6 \)
    Modifying factor = \( k = 1.0 \)
    \( f_a = \frac{140 \times 6}{1.0} = 840 \text{ psi} > 740 \text{ psi} \quad \text{Okay} \)
- Shear (AREMA 19-1.6.3.7)
  
  Bearing thickness = T > 2dₕ
  Design for dₕ = 3/4” to close gap at abutment during Level 1 seismic event
  
  T = 2 x ¾ = 1.5”  Try 2” thick (nominal) bearing
  
  Shear force = Fₛ = G dₕ A / T
  G = 180 psi (Figure 1-1c at 30 degrees F)
  Fₛ = 180 x ¾ 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 = dₑ = Σₑₑₑ < 1/8”
  LL+I stress = 67.7 / 144 = 0.47 ksi
  Comp strain = eₑ = 0.025  (Figure 1-1a)
  dₑ = 0.025 x 2.5 = 0.063” < 0.125”  Okay
Final bearing pad size = 2.5" (nominal) x 12" x 12" steel reinforced bearing

(2) EXTERNAL ELASTOMER LAYERS $\varnothing \frac{1}{4}''$
(5) STEEL LAYERS $\varnothing \ 0.075''$
(4) INTERNAL ELASTOMER LAYERS $\varnothing \frac{1}{2}''$

STEEL REINFORCED ELASTOMERIC BEARING
ABUTMENT DESIGN

Size the abutment for static loads.

ABUTMENT ELEVATION
Abutment Loads (AREMA 8-2.2.3)

- **Dead load**
  
  \[ P_v = 189 + 111 = 300 \text{ kips (Pg C-1)} \]
  
  \[ w_1 = 1 \times 7.5 \times 19 \times 0.15 = 21.4 \text{ kips} \]
  
  \[ w_2 = 3.5 \times 6 \times 19 \times 0.15 = 59.9 \text{ kips} \]
  
  \[ w_3 = 8 \times 2 \times 21 \times 0.15 = 50.4 \text{ kips} \]
  
  \[ w_4 = 1.75 \times 13.5 \times 2 \times 0.15 = 7.1 \text{ kips} \]
  
  Total = 439 kips

- **Live load**
  
  \[ P_v = 430 \text{ kips (Pg C-2)} \]

- **Longitudinal force**
  
  See sheet F-2 for longitudinal force calculations.

- **Earth pressure**
  
  \[ P_e = 0.036(19)(15.5)^2/2 = 82.2 \text{ kips} \]
  
  \[ P_{sc} = 0.3(16)(15.5) = 74.4 \text{ kips} \]
  
  Total = 157 kips

**ABUTMENT SECTION**

- **Service loads**
  
  \[ M_{dl} = 82.2 \times 15.5 / 3 + 74.4 \times 15.5 / 2 = 1001 \text{ k-ft} \]
  
  \[ M_{rl} = (300 + 430) \times 4 + 21.4 \times 5.75 + 59.9 \times 4.5 + 50.4 \times 4 + 7.1 \times 7.1 = 3565 \text{ k-ft} \]
  
  \[ x = (3565 - 1001) / 869 = 2.95 \text{ ft.} \]
  
  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

\[ P_{\text{max}} = \frac{869}{10} + \frac{869 \times 1.05}{20} = 133 \text{ kips} \]

\[ P_{\text{min}} = \frac{869}{10} - \frac{869 \times 1.05}{20} = 41 \text{ kips} \]

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

**TYPICAL PILE SECTION**

- Shear capacity (AREMA 8-35)

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

\[ \text{As} = 0.08 \text{ in}^2 \]

\[ \phi = 0.319" \]

\[ D' = 14 - 4 - 0.319 = 9.68" \]

\[ \phi V_s = 0.85 \times 0.08 \times 60 \times 9.68 \times 3.1416 / (2 \times 1.25) = 49.6 \text{ kips} \]

- Lateral pile analysis

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

Soil is sand with friction angle \( \phi = 32.5^\circ \)

Assume the piles are pinned at the top.

Assume soil modulus \( k = 90 \text{ pci} \) (LPILE users manual table 3.2)
Analysis results:

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

- Interaction diagram

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

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

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

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

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

$K_p = 7.7$

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

$K_1 = 585 / 3.52 = 166$ k/in
$K_2 = 2248 / 3.52 = 639$ 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.

$K_{pv} = 140 \times 2 \times 5 / 0.5" = 2800$ 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 = 100 - 41.8 = 58.2 kips

Abutment capacity will be controlled by shear capacity of backwall:

\[ \phi V_c = 0.85 \times 2 \times (4000)^{1/2} \times 19 \times 12 \times 9.5 / 1000 = 233 \text{ kips} \]

\[ P = 100 \times 233 / 58.2 = 400 \text{ kips} \]

• Abutment stiffness

Assume 2” maximum superstructure movement.  
Initial abutment stiffness = \( K_a = 397(2 - 0.75) / 2 = 248 \text{ k/in} \) (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.

**ABUTMENT STIFFNESS**

![Graph showing abutment stiffness](image)

The Caltrans abutment capacity for comparison purposes is \( 7.7 \times (6.67)^2 \times 19 / 8 = 814 \text{ kips} \) at 2% abutment movement = \( 0.02 \times 176 = 3.52” \) for an initial stiffness of \( 814 / 3.52 = 231 \text{ k/in} \).
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>
<thead>
<tr>
<th>Soil Type</th>
<th>Description</th>
<th>Site Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>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.</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>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.</td>
<td>1.2</td>
</tr>
<tr>
<td>3</td>
<td>20 to 40 feet of soft to medium-stiff clays with or without intervening layers of cohesionless soils.</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>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.</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The soil corresponds to type 2 with a site coefficient, \( S = 1.2 \)
• Damping adjustment factor (AREMA 9-1.4.3.2)

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

\[ \xi = \text{Percent critical damping} \]

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)

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

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

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

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 (tributary)
$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.
- Section properties:

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

Column: \[ A = 3.1416(1.5)^2 = 7.07 \text{ ft}^2 \]
\[ I_g = 3.1416(1.5)^4 / 4 = 3.98 \text{ ft}^4 \]
Assume \( I_e = 0.5 \times 3.98 = 1.99 \text{ ft}^4 \)

- Weight:

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

\[ W = 653 \text{ (page C-1)} + 0.15 \times (20 \times 19 + 7.07 \times 13.5) = 724 \text{ kips} \]

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.

   \[ K_{pv} = \frac{200 \times 2}{0.5} = 800 \text{ k/in (per pile)} \]

   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:

   \[ K_{ph} = 100 \text{ kips/in (per pile)} \]
   \[ K_{gh} = 100 \times 18 = 1800 \text{ k/in (group)} \]

   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.
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 \( \phi = 32.5 \) 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

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

\[
\alpha = 90^\circ
\]
\[
\beta = 0^\circ
\]
\[
\delta = 2 \times 32.5 / 3 = 21.7^\circ
\]

\[
K_p = 7.7
\]

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"

\[
K = 150 / 1.08 = 139 \text{ k/in}
\]

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.

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

![Diagram of bent analysis](image)

**ANALYSIS RESULTS**

<table>
<thead>
<tr>
<th>LOAD</th>
<th>ITEM</th>
<th>BENT CAP</th>
<th>LEFT COLUMN</th>
<th>RIGHT COLUMN</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_h = 100$ kips</td>
<td>AXIAL</td>
<td>-</td>
<td>-92 k</td>
<td>92 k</td>
</tr>
<tr>
<td></td>
<td>SHEAR</td>
<td>63 k</td>
<td>50 k</td>
<td>50 k</td>
</tr>
<tr>
<td></td>
<td>M+ve</td>
<td>377 k-ft</td>
<td>377 k-ft</td>
<td>397 k-ft</td>
</tr>
<tr>
<td></td>
<td>M-ve</td>
<td>377 k-ft</td>
<td>397 k-ft</td>
<td>377 k-ft</td>
</tr>
<tr>
<td></td>
<td>DEFL</td>
<td></td>
<td>0.202 in</td>
<td></td>
</tr>
</tbody>
</table>
2. The results of a transverse bent analysis are shown below for dead load, live load and impact.

![Diagram showing transverse load and moment](image)

### ANALYSIS RESULTS

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

- Natural period:

\[
T_m = 2\pi \sqrt{\frac{724}{386(307)}} = 0.491 \text{ seconds}
\]

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

Transverse \(C_m = \frac{0.317}{0.491^{2/3}} = 0.51 < 0.55 \quad \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.

\[ p(x) = C_m \cdot w(x) \]

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

1. Uniform lateral load on spans:

\[ p(x) = 0.51 \times 8.9 \text{ (page C-1)} = 4.54 \text{ k/ft} \]

2. Lateral load on bent:

\[ P = 0.51(724) = 369 \text{ kips} \]

3. Lateral load on abutments:

\[ P = 0.55(653 / 2) = 180 \text{ kips} \]

- 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**

<table>
<thead>
<tr>
<th>LOAD ( P_h = 369 \text{ kips} )</th>
<th>ITEM</th>
<th>BENT CAP</th>
<th>LEFT COLUMN</th>
<th>RIGHT COLUMN</th>
</tr>
</thead>
<tbody>
<tr>
<td>AXIAL</td>
<td>-</td>
<td>-339 k</td>
<td>339 k</td>
<td></td>
</tr>
<tr>
<td>SHEAR</td>
<td>232 k</td>
<td>185 k</td>
<td>185 k</td>
<td></td>
</tr>
<tr>
<td>M+ve</td>
<td>1391 k-ft</td>
<td>1391 k-ft</td>
<td>1465 k-ft</td>
<td></td>
</tr>
<tr>
<td>M-ve</td>
<td>1391 k-ft</td>
<td>1465 k-ft</td>
<td>1391 k-ft</td>
<td></td>
</tr>
<tr>
<td>DEFL</td>
<td>0.745 in</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
- 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)

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

   ii. Lateral pile stiffness (Page E-5)

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

   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

\[ \theta = \frac{1}{[(15.5 + 3.5 + 3) \times 12]} = 0.00379 \text{ radians} \]

\[ P1 = -P3 = -0.00379 \times 4 \times 12 \times 800 = -145 \text{ kips} \]

\[ K_{pr} = \frac{(145 \times 12 \times 4)}{22} = 316 \text{ k/in} \]

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

Capacity \[ P2 = 7.7 \times 1.5 \times 0.12 \times 3 \times (3 \times 24) = 299 \text{ kips} \]

Deflection \[ = 0.02 \times 4.5 \times 12 = 1.08" \]

\[ K = \frac{299}{1.08} = 277 \text{ k/in} \]

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.

\[ K = \frac{3EI}{L^3} \]

\[ K = \frac{3(3605)(1.99 \times 2 \times 20736)}{(17.5 \times 12)^3} = 96 \text{ kips/in} \]
C. The total longitudinal bent stiffness can be determined by summing up the foundation stiffness and the structure stiffness

\[ K = [\left(\frac{1800}{1} + \frac{277}{1} + \frac{316}{1} + \frac{96}{1}\right)]^{-1} = 71 \text{ k/in} \]

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

Capacity = 400 kips
\[ K_a = 248 \text{ kips/in} \]

3. Total longitudinal stiffness:

\[ K_a = 71 + 248 = 319 \text{ kips/in} \]

- Weight:

\[ W = 724 + 653 = 1377 \text{ kips} \]

- Natural period:

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

- Calculate the Longitudinal Seismic Response Coefficient (\( C_{lm} \)) for the structure (Page E-2)

\[ \text{Long } C_{lm} = \frac{0.253}{0.66^{2/3}} = 0.33 \leq 0.44 \quad \text{Use 0.33} \]
Perform static analysis on the bridge in the longitudinal direction.

1. Calculate the seismic load and deflection.
   \[ P = 0.33(1377) = 454 \text{ kips} \]
   \[ \delta = 454 / 319 = 1.42 \text{ inches} < 2" \text{ assumed in abutment stiffness (Page D-7)} \]

2. Distribute the seismic load between the abutment and bent based on the relative stiffness.
   Abutment load = \( \frac{454(248)}{319} = 353 \text{ kips} < 400 \text{ kips (Page D-7)} \) Okay
   Bent load = \( \frac{454(71)}{319} = 101 \text{ kips} \)

3. Determine the seismic loads in the bent columns.
   Longitudinal shear = \( V = 101 / 2 = 51 \text{ kips} \)
   Longitudinal moment = \( M = 101 \times 17.5 / 2 = 884 \text{ k-ft} \)
- 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:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>Earthquake</th>
<th>Dead Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEFT COLUMN</td>
<td>RIGHT COLUMN</td>
</tr>
<tr>
<td>AXIAL</td>
<td>-339 k</td>
<td>339 k</td>
</tr>
<tr>
<td>SHEAR</td>
<td>185 k</td>
<td>185 k</td>
</tr>
<tr>
<td>M_{+ve}</td>
<td>1391 k-ft</td>
<td>1465 k-ft</td>
</tr>
<tr>
<td>M_{-ve}</td>
<td>1465 k-ft</td>
<td>1391 k-ft</td>
</tr>
</tbody>
</table>

- Longitudinal column loads:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>Earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEFT COLUMN</td>
</tr>
<tr>
<td>AXIAL</td>
<td>-</td>
</tr>
<tr>
<td>SHEAR</td>
<td>51 k</td>
</tr>
<tr>
<td>M_{top}</td>
<td>-</td>
</tr>
<tr>
<td>M_{bot}</td>
<td>884 k-ft</td>
</tr>
</tbody>
</table>

- 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.

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

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

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

  \[ V = \frac{66}{2} = 33 \text{ kips} \]
  \[ M = \frac{66 \times 19}{2} = 627 \text{ k-ft} \]

  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.

\[ W = 0.045 \times 7.5 \times 70 = 24 \text{ kips} \]

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.

\[ WL = 0.3 \times 70 = 21 \text{ kips} \]

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

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

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

- Longitudinal reinforcing design

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

  \[ A_s = 32 \text{ in}^2 \]
  \[ \rho_g = 0.031 \]
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).

### INTERACTION DIAGRAM

<table>
<thead>
<tr>
<th>c</th>
<th>Mn</th>
<th>Pn</th>
<th>Reinf Tension</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-1920</td>
<td>-1920</td>
<td></td>
<td>Maximum tension</td>
</tr>
<tr>
<td>1.8</td>
<td>72.3</td>
<td>-1869.2</td>
<td>-1920</td>
<td></td>
</tr>
<tr>
<td>3.6</td>
<td>417.8</td>
<td>-1579.4</td>
<td>-1721</td>
<td></td>
</tr>
<tr>
<td>5.4</td>
<td>862.1</td>
<td>-1168.6</td>
<td>-1493</td>
<td></td>
</tr>
<tr>
<td>7.2</td>
<td>1230.2</td>
<td>-789.4</td>
<td>-1352</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>1536.8</td>
<td>-426.8</td>
<td>-1232</td>
<td></td>
</tr>
<tr>
<td>9.23</td>
<td>1573.2</td>
<td>-377.4</td>
<td>-1215</td>
<td></td>
</tr>
<tr>
<td>9.26</td>
<td>1577.9</td>
<td>-370.9</td>
<td>-1212</td>
<td></td>
</tr>
<tr>
<td>11.13</td>
<td>1829.1</td>
<td>1.4</td>
<td>-1095</td>
<td>Pure bending</td>
</tr>
<tr>
<td>11.3</td>
<td>1851.8</td>
<td>36.1</td>
<td>-1084</td>
<td></td>
</tr>
<tr>
<td>13.02</td>
<td>2029.4</td>
<td>372.5</td>
<td>-980</td>
<td></td>
</tr>
<tr>
<td>16.32</td>
<td>2224.2</td>
<td>1015.5</td>
<td>-782</td>
<td></td>
</tr>
<tr>
<td>16.82</td>
<td>2235.8</td>
<td>1115.4</td>
<td>-750</td>
<td></td>
</tr>
<tr>
<td>16.85</td>
<td>2236.2</td>
<td>1121.6</td>
<td>-747</td>
<td></td>
</tr>
<tr>
<td>17.09</td>
<td>2239.6</td>
<td>1170.5</td>
<td>-731</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>2219.3</td>
<td>1566.6</td>
<td>-587</td>
<td>Balanced strain</td>
</tr>
<tr>
<td>0</td>
<td>4481.2</td>
<td></td>
<td></td>
<td>Maximum compression</td>
</tr>
</tbody>
</table>
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) \( \rho_s > 0.12 \frac{f'_c}{f_y} = 0.12 \times \frac{4}{60} = .008 \)
(2) \( \rho_s \geq \) that required by Chapter 8, Section 2.11.2

(1) controls by inspection

\[
\text{Volumetric ratio} = \rho_s \times \frac{A_s}{D \times s} \\
\text{Assume #5 spiral:} \quad 0.008 < 4 \times \frac{0.31}{(32 \times s)} \\
s < 4.84''
\]

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 \text{ mm})

(1) controls for maximum clear spacing between spirals

\[
\text{Clear s} = 3.5 - .625 = 2.875'' < 3'' \quad \text{Therefore 3.5'' spacing is okay}
\]

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 = $13.5 \times 12 / 6 = 27”$
  (3) $l_o \geq 18”$ (450 mm)

  (1) 36” controls plastic hinge length
• Use iterative hand analysis method

Total weight: \( W = 371 \times 2 = 742 \text{ kips} \) (Page E-8)
Center of load: \( Y_{cg} = 5.5 \text{ ft} \) (Page E-4)

Initial axial loads: \( P_l = 371 \text{ kips} \) (Page E-8)
\( P_r = 371 \text{ kips} \)

Initial nominal moments: \( M_l = 2029 \text{ k-ft} \) (Page F-4)
\( M_r = 2029 \text{ k-ft} \)

Total lateral load: \( V = 2 \times 1.3 \frac{(2029 + 2029)}{13.5} = 782 \text{ kips} \)

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

Continue iterations until \( V \) converges:

<table>
<thead>
<tr>
<th>No</th>
<th>( P_l )</th>
<th>( P_r )</th>
<th>( M_l )</th>
<th>( M_r )</th>
<th>( V )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-427</td>
<td>1169</td>
<td>1537</td>
<td>2240</td>
<td>727</td>
</tr>
<tr>
<td>2</td>
<td>-372</td>
<td>1114</td>
<td>1578</td>
<td>2236</td>
<td>735</td>
</tr>
<tr>
<td>3</td>
<td>-379</td>
<td>1121</td>
<td>1573</td>
<td>2236</td>
<td>734</td>
</tr>
</tbody>
</table>

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: \( P_{\min} = 371 - 2.68 \times 339 = -538 \text{ kips} \)
\( P_{\max} = 371 + 2.68 \times 339 = 1280 \text{ kips} \)

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

Moment: \( M_{r\text{top}} = 72 + 1391 \times 2.68 = 3800 \text{ k-ft} \)
\( M_{r\text{bot}} = 35 + 1465 \times 2.68 = 3961 \text{ k-ft} \)
\( M_{l\text{top}} = 72 - 1391 \times 2.68 = -3656 \text{ k-ft} \)
\( M_{l\text{bot}} = 35 - 1465 \times 2.68 = -3891 \text{ k-ft} \)
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.

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

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

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

Therefore the concrete shear strength, \( \phi V_c \), is assumed to be zero for the column in tension.

3. AREMA 8-2.35.2 - Concrete shear strength

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

Shear reinforcing strength \( \phi V_s > 431 - 88 = 343 \text{ kips} \)
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"

\[
D' = 36 - 4 - 0.625 = 31.4''
\]

\[
s = 3.5''
\]

\[
\phi V_s = 0.85 \times 0.31 \times 60 \times 31.4 \times 3.1416 / (2 \times 3.5) = 222 \text{ kips} < 343 \text{ NG}
\]

Increase confinement reinforcement:

Try #6 spiral @ 3"

\[
\phi V_s = 0.85 \times 0.44 \times 60 \times 31.4 \times 3.1416 / (2 \times 3) = 369 \text{ kips} > 343 \text{ OK}
\]

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

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

- Flexural design per AREMA 8-2.32

  Positive moment
  
  $M_u \text{+ve} = 1825 \text{ k-ft}$
  $b = 60", h = 48", d = 44"$
  $A_{\text{+ve}} > 9.52 \text{ in}^2$

  Negative moment
  
  $M_u \text{-ve} = 1885 \text{ k-ft}$
  $b = 60", h = 48", d = 44"$
  $A_{\text{-ve}} > 9.84 \text{ in}^2$

  AREMA 9-1.4.7.3.2b(3)
  Add 8% column steel
  $0.08 \times 32 = 2.56 \text{ in}^2$
  Total $A_{\text{+ve}} = 12.08 \text{ in}^2$

  AREMA 9-1.4.7.3.2b(3)
  Add 8% column steel
  $0.08 \times 32 = 2.56 \text{ in}^2$
  Total $A_{\text{-ve}} = 12.4 \text{ in}^2$
Try (10) #10 top and bottom, $A_s = 12.7 \text{ in}^2 > 12.4$  Okay

BENT CAP SECTION

- Shear design per AREMA 8-2.35

\[ V_u = 854 \text{ kips} \]
\[ b = 60" , \ h = 48" , \ d = 44" \]

\[ \phi V_c = 0.85 \times 2 \left(4000 \right)^{1/2} \times 60 \times 44 / 1000 = 284 \text{ kips} \]

\[ \phi V_s > 854 - 284 = 570 \text{ kips} \]
Assume 4 legs of #6 stirrups

\[ s < 0.85 \times 4 \times .44 \times 60 \times 44 / 570 = 6.92" \text{ use 6"} \]

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):

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

Inside the joint:

\[ A_v > 0.08 \times 32 = 2.56 \text{ in}^2 \]
\[ \text{no. of 4-legged #6 stirrups} = \frac{2.56}{0.44 \times 4} = 1.5 \text{ say 2} \]

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

\[ l_{dh} \geq \text{that required by Chapter 8, Section 2.17} \]
\[ l_{dh} \geq 8d_b \]
\[ l_{dh} \geq 6" \]
\[ l_{dh} \geq \frac{f_y d_b}{65 \sqrt{f_c}} \text{ inches} \]

For straight bars:

\[ l_d \geq \text{that required by Chapter 8, Sections 2.14 through 2.16} \]
\[ l_d \geq 2.5 \text{ times that required in this Article for hooked bars in tension} \]

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) \[ l_d = 0.04 \times 1.0 \times 60000 \times 0.75 / (4000)^{1/2} = 28" \text{ (AREMA 8-2.14)} \]

(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) -

\[ \rho_s > 0.4 \times \frac{A_s}{(l_d)^2} \]
\[ \rho_s = 0.4 \times \frac{32}{(41)^2} = 0.0076 \]

Volumetric ratio = \( \rho_s = 4 \times \frac{A_s}{(D \times s)} \)

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:

\[ v_c = 20 \sqrt{f'_c} \text{ psi} \]
\[ \phi V_c = \phi 20 \sqrt{f'_c} bd \]
\[ \phi V_c = 0.85 \times 20 \times (4000)^{1/2} \times 60 \times 44 / 1000 = 2838 \text{ kips} \]

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.

\[ T = 1095 \text{ kips} \]
\[ V_u = 1095 \times 1.3 = 1424 \text{ kips} < 2838 \text{ kips} \quad \text{Okay} \]
• 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:

\[ W = 24 \times 12 \times 3 \times 0.15 + (24 \times 12 - 7.07 \times 2)(1.5 \times 0.12) = 179 \text{ kips} \]

- Service loads: D + L

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

\[ P_l = P_r = 371 + 342 = 713 \text{ kips} \]

Column shears and moment cancel each other out.

\[ P_1 = (713 \times 2 + 179) / 12 = 134 \text{ kips} \]

PILE CAP TRANSVERSE FREE BODY DIAGRAM

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

\[ P_l = 1.4[371 + 1.67(342)] = 1319 \text{ kips} \quad P_r = 1319 \text{ kips} \]

\[ V_l = -1.4[7 + 1.67(7)] = -26 \text{ kips} \quad V_r = 26 \text{ kips} \]

\[ M_l = -1.4[35 + 1.67(35)] = -131 \text{ k-ft} \quad M_r = 131 \text{ k-ft} \]

\[ P_1 = (1319 \times 2 + 1.4 \times 179) / 12 = 241 \text{ kips} \]
1. Transverse EQ load combination is shown on page F-7 and F-8 for the column plastic hinging load.

\[
\begin{align*}
P_l &= -379 \text{ kips} & P_r &= 1121 \text{ kips} \\
V_l &= 303 \text{ kips} & V_r &= 431 \text{ kips} \\
M_l &= 1.3 \times 1573 = 2045 \text{ k-ft} & M_r &= 1.3 \times 2236 = 2907 \text{ k-ft}
\end{align*}
\]

Resultant loads:

\[
\begin{align*}
P &= 1121 - 379 + 179 = 921 \text{ kips} \\
V &= 303 + 431 = 734 \text{ kips}, V_p = 734 / 18 = 41 \text{ kips/pile} \\
M &= 2045 + 2907 + 734 \times 3 = 7154 \text{ k-ft}
\end{align*}
\]

\[
I_{\text{piles}} = 6(2)^2 + 6(6)^2 + 6(10)^2 = 840
\]

\[
P_1 = 921 / 18 - 7154 \times 10 / 840 = -34 \text{ kips} \\
P_6 = 921 / 18 + 7154 \times 10 / 840 = 136 \text{ kips}
\]

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.

\[
P = 371 \times 2 + 179 = 921 \text{ kips} \\
1.3 \times M_n = 1.3 \times 2029 = 2638 \text{ k-ft} \\
V = 2638 \times 2 / (5.5 + 13.5) = 278 \text{ kips} \\
M = 2638 \times 2 + 278 \times 3 = 6110 \text{ k-ft}
\]

\[
P_1 = 921 / 18 - 6110 / (6 \times 8) = -76 \text{ kips} \\
P_3 = 921/ 18 + 6110 / (6 \times 8) = 178 \text{ kips}
\]
- Final pile design loads

Service:

\[ P = 134 \text{ kips} \]

Seismic:

\[ P = 178 \text{ kips} \]
\[ P = -76 \text{ kips (tension)} \]

Design pile lengths for 70 ton capacity under service loads and 100 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.

\[
\begin{align*}
\text{PLAN - LATERAL BENDING OF GIRDERS} \\
\text{• Lateral bending of girders (Level 1)} \\
\text{w} &= 4.54 \text{ k/ft (Page E-9)} \\
\text{V} &= 4.54 \times 70 / 2 = 159 \text{ kips} \\
\text{M} &= 4.54 \times (70)^2 / 8 = 2781 \text{ k-ft} \\
\text{Prestressing load (Page C-3):} \\
\text{P} &= 947 \times 4 = 3788 \text{ kips} \\
\text{e} &= 26.08" \\
\text{Dead load (Page C-1):} \\
\text{V} &= 189 + 111 = 300 \text{ kips (4 girders)} \\
\text{M} &= 3189 + 1879 = 5068 \text{ k-ft (4 girders)} \\
\text{Section properties (Page C-3):} \\
\text{A} &= 1316 \times 4 = 5264 \text{ in}^2 \\
\text{I}_x &= 678151 \times 4 = 2713000 \text{ in}^4 \\
\text{I}_y &= 312359 \times 4 + 1316 \times 2 \times 2 (42)^2 = 10535000 \text{ in}^4 
\end{align*}
\]
Bending stress:

- Conservatively assume that the girders act separately

\[
\sigma_{(DL+PS)} = \frac{947}{5264} - \frac{(5068 \times 12 - 3788 \times 26.08)}{2713000} = 0.642 \text{ ksi}
\]

\[
\sigma_{EQ} = \frac{2781 \times 12 \times 21}{4 \times 312359} = 0.561 \text{ ksi} < 0.642 \text{ ksi} \quad \text{Okay}
\]

(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 \times 28 = 840 \text{ in}^2

  - Transverse seismic load (Level 1):

    \[ V = 180 \text{ kips (Page E-9)} \]
    \[ M = 180 \times 2 / 2 = 180 \text{ k-ft} \]
- Shear friction reinforcement design (AREMA 8-2.35.4):

  Maximum shear resistance = 0.85 \times 0.8 \times 840 = 571 \text{ kips} > 180 \text{ kips}  \text{ Okay}

  \[ A_{vf} > \frac{180}{0.85 \times 60 \times 1.0} = 3.53 \text{ in}^2 \]

  Flexural reinforcement:

  \[ b = 30", \quad d = 28 - 2.5 = 25.5", \quad A_s > 1.65 \text{ in}^2 \]

  At abutments, use (6) #5  \[ A_s = 1.86 > 1.65 \text{ Okay} \quad A_{vf} = 3.72 > 3.53 \text{ Okay} \]

  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.

\[ P = 3.72 \times 60 \times 1.4 = 312 \text{ kips} \]

1. Abutment reinforcement

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

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

\[ A_s > 230 / (0.9 \times 60) = 4.26 \text{ in}^2 \]

Use (6) #8 at the abutment seat

\[ A_s = 4.74 \text{ in}^2 > 4.26 \text{ in}^2 \text{ Okay} \]

2. Bent cap reinforcement

\[ V = 2 \times 230 = 460 \text{ kips} \]

\[ A_s > 460 / (0.9 \times 60) = 8.52 \text{ in}^2 \]

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

\[ A_s = 12.7 \text{ in}^2 > 8.52 \text{ in}^2 \text{ Okay} \]
• 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):

  \[ V = 101 \times 0.59 / 0.22 = 270 \text{ kips} \] (Page E-13)

- Shear pin design

  \[ P = 270 / 8 = 33.8 \text{ kips} \]
\[ P = \frac{\pi d^2 f_y}{4\sqrt{3}} \]

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, \( f_y = 130 \text{ ksi} \)

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

Pad thickness = 2.9" (Page C-11)
\[ M = PL / 2 = 33.8 \times 2.9 / 2 = 49 \text{ k-in} \]
\[ Z = (1.5)^3 / 6 = 0.56 \text{ in}^3 \]
\[ f_b = 49 / 0.56 = 88 \text{ ksi} < 130 \text{ ksi} \quad \text{Okay} \]