Appendix A
### TOTAL ESTIMATED BRIDGE QUANTITIES - BOTH BRIDGES

<table>
<thead>
<tr>
<th>ITEM NO.</th>
<th>ITEM CODE</th>
<th>ITEM DESCRIPTION</th>
<th>UNIT</th>
<th>WESTBOUND BRIDGE</th>
<th>EASTBOUND BRIDGE</th>
<th>TOTAL</th>
<th>AS BUILT QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>230-073500</td>
<td>CONCRETE PLACED IN CONCRETE</td>
<td>YD</td>
<td>830</td>
<td>835</td>
<td>1665</td>
<td>1665</td>
</tr>
<tr>
<td>2</td>
<td>2402-272000</td>
<td>EXCAVATION, CL 20</td>
<td>YD</td>
<td>156</td>
<td>156</td>
<td>312</td>
<td>312</td>
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<tr>
<td>3</td>
<td>2401-010900</td>
<td>STRUCTURE (B3015)</td>
<td>CY</td>
<td>533</td>
<td>538</td>
<td>1071</td>
<td>1071</td>
</tr>
<tr>
<td>4</td>
<td>2404-075003</td>
<td>PIPE, STEEL, DIAMETER 36 IN</td>
<td>FT</td>
<td>26,240</td>
<td>26,240</td>
<td>52,480</td>
<td>52,480</td>
</tr>
<tr>
<td>5</td>
<td>2404-077405</td>
<td>REINFORCEMENT, STRETCH COATED</td>
<td>LB</td>
<td>90,255</td>
<td>90,255</td>
<td>180,510</td>
<td>180,510</td>
</tr>
<tr>
<td>6</td>
<td>2407-086100</td>
<td>BEAM, FIP, 129000</td>
<td>EACH</td>
<td>12</td>
<td>12</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>7</td>
<td>2407-085000</td>
<td>BEAM, FIP, 129000</td>
<td>EACH</td>
<td>12</td>
<td>12</td>
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<tr>
<td>8</td>
<td>2401-010900</td>
<td>STRUCTURE (B3015)</td>
<td>CY</td>
<td>533</td>
<td>538</td>
<td>1071</td>
<td>1071</td>
</tr>
<tr>
<td>9</td>
<td>3001-003000</td>
<td>POLYMER CONCRETE</td>
<td>YD</td>
<td>100</td>
<td>100</td>
<td>200</td>
<td>200</td>
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<tr>
<td>10</td>
<td>3001-003000</td>
<td>POLYMER CONCRETE</td>
<td>YD</td>
<td>100</td>
<td>100</td>
<td>200</td>
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<td>YD</td>
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<td>100</td>
<td>200</td>
<td>200</td>
</tr>
</tbody>
</table>

### ESTIMATE REFERENCE INFORMATION

- Includes: Fencing, draining, and other miscellaneous construction work.
- Does not include: Placing and finishing concrete, reinforcing steel, and other materials.
- Note: Refer to the construction plans for specific details.

### GENERAL NOTES

- This bridge is designed for loads of 10,000 lbs at the specified locations.
- The bridge is designed for a 20-year bridge service life.
- The bridge is designed for a traffic capacity of 50,000 vehicles per day.

### TRAFFIC CONTROL PLAN ON RELOCATED U.S. 34

Note: This bridge is being constructed on a traffic lane closure. Traffic will be controlled by posted signs and traffic control devices. This traffic control plan is for reference only.

### TRAFFIC CONTROL PLAN ON IOWA V43

Note: The roadway will be closed to thru traffic during the bridge construction period. Traffic will be controlled by posted signs and traffic control devices. This traffic control plan is for reference only.

### DESIGN STRESSES

- The design of this bridge is based on the following materials and specifications:
  - Reinforcing steel: ASTM A615 Grade 60
  - Prestressed concrete: ASTM A416

### SPECIFICATIONS

- Design: 1%=3,000 lb-ft
- Material: Prestressed concrete beams
- Load: 50,000 vehicles per day
- Service life: 50 years

---

This bridge has been designed and constructed by the Iowa Department of Transportation, Highway Division.
PIER NOTES:

ALL EXPOSED CORNERS OF 30° OR SHAPER ARE TO BE FILLED WITH A 2" SLUGGISH AND NEUTRAL SLUG.

MINIMUM CLEAR DISTANCE FROM FACE OF CONCRETE TO NEAR REINFORCING BAR IS TO BE 2 INCHES UNLESS OTHERWISE NOTED ON SHEET. THE 90° FOOTING TO COLUMN INSELS ARE TO BE IN PLACE BEFORE PLACING CONCRETE IS PLACED.

THE COLUMN CONCRETE NUMBERS IN THE SHEET TO MOLD AND FORM ARE SHOWN AS THE LENGTH OF THE SPIRAL SHOWN DOES NOT INCLUDE THE LAMPS LENGTH OF THE SPIRALS. THE COST OF THE LAMPS AT SPACERS IS TO BE INCLUDED IN THE PRICE BID FOR OTHER REINFORCEMENT.

FOR ANY TIES GRADERS AT 3/8" CENTERS MAY BE SUBSTITUTE FOR THE SPHERICAL REINFORCEMENT. PAYMENT WILL BE BASED ON THE WEIGHT OF SPHERICAL REINFORCEMENT. NO ALLOWANCE FOR REINFORCING STEEL AT WEIGHT WILL BE ALLOWED. SEE BENT BAR DETAILS FOR SPHERE LAY LAPS.

THE DESIGN BEARING FOR PIER FILES IS 44 TONS.

CONCRETE PLACEMENT QUANTITIES

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>PIER NO.</th>
<th>PROD. NO.</th>
<th>QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>FOOTING</td>
<td>317</td>
<td>2I4</td>
<td>53.0</td>
</tr>
<tr>
<td>COLUMN</td>
<td>318</td>
<td>319</td>
<td>22.0</td>
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<tr>
<td>CAST-IN-PLACE</td>
<td>319</td>
<td>320</td>
<td>46.5</td>
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<tr>
<td>TOTAL CAST</td>
<td></td>
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<td>121.5</td>
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ESTIMATED QUANTITIES - WESTBOUND BRIDGE

<table>
<thead>
<tr>
<th>ITEM</th>
<th>UNITS</th>
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<tr>
<td>STRUCTURAL CONCRETE (CONCRETE)</td>
<td>CUBIC YD</td>
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<td>LBS</td>
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<td>LFS</td>
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<tr>
<td>WALL 99'4&quot; WALL (REINFORCING)</td>
<td>LFS</td>
<td>2,940.0</td>
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<tr>
<td>ELEVATION PEDIMENT (REINFORCING)</td>
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</tr>
<tr>
<td>WALL 100'6&quot; LONGITUDINAL (REINFORCING)</td>
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</table>

DUAL 216'-0" X 40' PRETENSIONED PRESTRESSED CONCRETE BEAM BRIDGES
50'-0" END SPAN 116'-6" INTERIOR SPAN

WESTBOUND BRIDGE

DESIGN FOR 50'-0" SPAN

WAPELO COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - HIGHWAY DIVISION

DESIGN NO. 2005-07-36

WAPELO COUNTY

DESIGN NO. 2005-07-36

WAPELO COUNTY

DESIGN NO. 2005-07-36
PIER NOTES:
- ALL EXPOSED CORNERS OF 20° OR SHAPE ARE TO BE FILLED WITH A 2" UNPLUGGED AND REBARDED SPUR.
- MINIMUM CLEAR DISTANCE FROM FACE OF CONCRETE TO NEAR REINFORCING BAR IS TO BE 2 INCHES UNLESS OTHERWISE NOTED ON SHEET. THE 90° FOOTING TO COLUMN SPACING IS TO BE 2'-0" PLACE BEFORE FACING COMPLETE IS PLACED.
- THE DESIGN BEARING FOR PIER PILES IS 44 TOUS.
- THE DESIGN BEARING FOR PIER PILES IS 44 TOUS.

CONCRETE PLACEMENT QUANTITIES

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>PIER NO.</th>
<th>PUB NO.</th>
<th>QUANTITY</th>
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</thead>
<tbody>
<tr>
<td>FOOTING</td>
<td>3,5</td>
<td>3,5</td>
<td>630</td>
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<tr>
<td>COLUMN</td>
<td>3,8</td>
<td>3,8</td>
<td>130</td>
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<tr>
<td>CAST &amp; JUPES</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
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<tr>
<td>TOTAL CEMENT</td>
<td>83.8</td>
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ESTIMATED QUANTITIES - EASTBOUND BRIDGE

<table>
<thead>
<tr>
<th>ITEM</th>
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<th>QUANTITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>STRUCTURAL CONCRETE (BRIDGE)</td>
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<td>REINFORCING STEEL</td>
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<td></td>
</tr>
<tr>
<td>HEAT-RESISTANT CONCRETE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BEARING PLANE</td>
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REINFORCING BAR LIST - ONE PIER

<table>
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<th>SHAPE</th>
<th>NO.</th>
<th>LENGTH</th>
<th>WEIGHT</th>
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<td>48</td>
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<tr>
<td>5a</td>
<td>CAP, SIDES, LONG.</td>
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<td>48</td>
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<td>CAP, SIDES, LONG.</td>
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<td>9.35</td>
</tr>
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<td>4b</td>
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<td>14</td>
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<td>0.11</td>
</tr>
<tr>
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<td>10</td>
<td>1008</td>
<td>1008</td>
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<tr>
<td>6c</td>
<td>CAP, HOOPS, LONG.</td>
<td>35</td>
<td>14</td>
<td>1406</td>
<td>1406</td>
</tr>
<tr>
<td>6d</td>
<td>CAP, HOOPS, LONG.</td>
<td>4</td>
<td>10</td>
<td>42</td>
<td>42</td>
</tr>
<tr>
<td>4a</td>
<td>COLUMN SPINDLE</td>
<td>5415</td>
<td>3</td>
<td>141.7</td>
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<tr>
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<td>FOOTING TO COLUMN SPINDLE</td>
<td>5541</td>
<td>3</td>
<td>141.7</td>
<td>141.7</td>
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<tr>
<td>5a</td>
<td>COLUMN TIE</td>
<td>22</td>
<td>22</td>
<td>2996</td>
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<td>5a</td>
<td>REBAR OPTIONS</td>
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<td>10</td>
<td>271</td>
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<td>5a</td>
<td>TIE BAR</td>
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<td>0.01</td>
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</tr>
<tr>
<td>5a</td>
<td>TIE BAR</td>
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<td>6039</td>
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</tr>
</tbody>
</table>

BENT BAR DETAILS

5c1, 5c2,5c3, 5c4 & 5c5

5ni

ALTERNATE COLUMN TIE

NOTE: ALL DIMENSIONS ARE OUT TO OUT, D - DIAMETER, L - LENGTH, L/2, 3/4 L/2, 3/8 L/2.
ABUTMENT NOTES:

MINIMUM CLEAR DISTANCE FROM FACE OF CONCRETE TO NEAR
REINFORCING BAR IS TO BE 2" UNLESS OTHERWISE NOTED OR SHOWN.
THE DESIGN BEARING FOR THE ABUTMENT PLATES IS 40 TONS.

PART REAR ELEVATION AT ABUTMENT

PART SECTION A - A

NOTE: 1. NO PANS IN R.A.S. 108.2008
2. S.W. PANS PARALLEL TO LONGITUDINAL STEEL.

PART SECTION B - B

PART SECTION C - C

ABUTMENT EXCAVATION DETAILS

DUAL 218'-0" X 40' PRETENSIONED PRESTRESSED CONCRETE BEAM BRIDGES
50' 0" SPAN - 150'-6" INTERIOR SPAN
ABUTMENT DETAILS

WAPELLO COUNTY
IOWA DEPARTMENT OF TRANSPORTATION - HIGHWAY DIVISION
DESIGNED BY KRAMER & EICHENHOLZ, INC. DECEMBER 3, 2002
DESIGN NO. 385
### Table of Top of Slab Elevations - Westbound Bridge

| PIER NO. | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |

**Note:** Add 0.00 to all elevations.

---

### Slab Thickness Details

Note: The slab thickness at bearings is based on the anticipated moment concentrations at tops of the bearings, and is not guaranteed for construction.

**Design:** 218'-0" x 40' Precast Prestressed Concrete Beam Bridges

Superstructure Details - Westbound

**Project:** Iowa Department of Transportation - Highway Division

**County:** Wood

**Sheets:** 19 of 36

**Sheet:** Drawn by: 07/05/03

---

### Beam Camber Data

- Beam 1: -0.150
- Beam 2: -0.150
- Beam 3: -0.150
- Beam 4: -0.150

**Note:** Used to determine beam seat elevations.
### Table of Beam Line Haunch Elevations - Westbound Bridge

<table>
<thead>
<tr>
<th>Beam Line</th>
<th>R Pier #1 Elevations</th>
<th>L Pier #2 Elevations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Line 1</td>
<td>15.70</td>
<td>15.70</td>
</tr>
<tr>
<td>Beam Line 2</td>
<td>15.70</td>
<td>15.70</td>
</tr>
<tr>
<td>Beam Line 3</td>
<td>15.70</td>
<td>15.70</td>
</tr>
</tbody>
</table>

### Notes:
- Add 0.001 to all elevations.

### Miscellaneous Data Table

<table>
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<tr>
<th>Anticipated</th>
<th>R Cross Slope</th>
<th>L Cross Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1/4</td>
<td>1/4</td>
</tr>
</tbody>
</table>

### Haunch Detail

- **Notes:**
  - Bridge seat elevations are set based on theoretical camber and beam deflection. These adjusted seats will provide a theoretical beam match within design parameters. Actual haunches are determined using surveyed top of beam elevation and beam line.
  - Haunch elevations data, allowable deflection, and deflection limits are shown in the table.

### HAUNCH DETAIL

- **Note:** Haunch locations are at the same location as the encircled letters and numbers shown on Design Sheet 18.

---

**Design Sheet 18**

- **DUAL 217'-0 x 40' Prestressed Concrete Beam Bridges**
- **50'-6 END SPAN**
- **10'-0 INTERIOR SPAN**
- **Superscript Detail**
- **Westbound Bridge**
- **Design Sheet 18 of 36**
- **West Branch**

---

**Engineer:**

- West Branch 2009-07-21
- J. L. Smith
- Project No: 000040049499
- Final Report No: 6803

**Design:**

- J. L. Smith
- Project No: 000040049499
- Final Report No: 6803
### TABLE OF BEAM LINE HAUNCH ELEVATIONS - EASTBOUND BRIDGE

| Length | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 |
|--------|---|---|---|---|---|---|---|---|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| Beam Line 1 | 15.63 | 15.63 | 15.81 | 15.80 | 15.78 | 15.75 | 15.73 | 15.72 | 15.71 | 15.70 | 15.69 | 15.68 | 15.67 | 15.66 | 15.65 | 15.64 | 15.63 | 15.62 | 15.61 | 15.60 | 15.59 | 15.58 | 15.57 |
| Beam Line 2 | 15.49 | 15.49 | 15.67 | 15.66 | 15.64 | 15.62 | 15.60 | 15.59 | 15.58 | 15.57 | 15.56 | 15.55 | 15.54 | 15.53 | 15.52 | 15.51 | 15.50 | 15.49 | 15.48 | 15.47 | 15.46 | 15.45 | 15.44 |
| Beam Line 3 | 15.35 | 15.35 | 15.53 | 15.52 | 15.50 | 15.48 | 15.46 | 15.45 | 15.44 | 15.43 | 15.42 | 15.41 | 15.40 | 15.39 | 15.38 | 15.37 | 15.36 | 15.35 | 15.34 | 15.33 | 15.32 | 15.31 | 15.30 |
| Beam Line 4 | 15.21 | 15.21 | 15.39 | 15.38 | 15.36 | 15.34 | 15.32 | 15.31 | 15.30 | 15.29 | 15.28 | 15.27 | 15.26 | 15.25 | 15.24 | 15.23 | 15.22 | 15.21 | 15.20 | 15.19 | 15.18 | 15.17 | 15.16 |

### MISCELLANEOUS DATA TABLE

| Beam Line | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 |
|-----------|---|---|---|---|---|---|---|---|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| Anticipated Field Variation to Slab (mm) | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 |
| Cross Slope Allowable Values (mm) | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 | 50 |
| Allowable Field Variation | MAX. | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL | ALL |

### HAUNCH DETAIL

**NOTE 1:**
- To calculate field variation required at each location, survey the beam top location (as shown on the "Top of Slab Elevation") on Design Sheet 2D. Subtract the surveyed beam from the Beam Line 1 Haunch elevation. This value will be the Haunch required (see "Field Haunch in Haunch Detail").
- The Beam Line 1 Haunch Elevations includes adjustments for slab thicknesses and anticipated deflections. No additional calculations are necessary. The maximum and minimum elevations indicated in the Miscellaneous Data Table are adjustments to the grade on additional Haunch measurement will be required.

**NOTE 2:**
- The Haunch Locations are at the same location as the encircled letters and numbers shown on Design Sheet 20.

**DESIGN FOR 20 YEAR FLOOD:**
- Dual 216x6 x 40-Pretensioned Prestressed Concrete Beam Bridges
- 105-6 End Span
- 115-6 Interior Span
- Structural Details - Eastbound
- Iowa Department of Transportation - Highway Division

**DESIGNED BY:**
- [Design Firm]

**CHECKED BY:**
- [Checking Firm]

**PROJECT NO:**
- [Project Number]
BARRIER RAIL NOTES:
- Minimum clear distance from edge of concrete to rear reinforcing bar is to be 2" unless otherwise noted on plans.
- All reinforcing bars at a minimum spacing of 1/2 ft. in fact and construction joint reinforcement is to be spaced between vertical bars at a minimum spacing of 1/2 ft. Construction joint contact surface shall be sealed in accordance with construction specifications.
- All reinforcing bars shall be cut to length with a minimum of 2" of bars extending beyond construction joint.

PART ELEVATION VIEW
BARRIER RAIL JOINT DETAILS

PART SECTION F-F

BARRIER RAIL Quantities:
- Standard Section: 462 ft. of barrier rail per ft.
- Eastbound Bridge: Total 2 ft.
- Westbound Bridge: Total 2 ft.
- Total: 4 ft. of barrier rail

CONCRETE PLACEMENT SUMMARY
- Total 4 ft. of barrier rail
- Eastbound Bridge: Total 2 ft.
- Westbound Bridge: Total 2 ft.
- Total: 4 ft. of barrier rail

CONCRETE BARRIER RAIL Quantities
- Standard Section: 462 ft. of barrier rail per ft.
- Eastbound Bridge: Total 2 ft.
- Westbound Bridge: Total 2 ft.
- Total: 4 ft. of barrier rail

BENT BAR DETAILS
- All dimensions are out to cut, D = 1" pipe diameter.
TRAFFIC CONTROL PLAN

1. Traffic will be maintained on U.S. 12 and all others.
   U.S. 12 will be closed to traffic.
   Traffic will be detoured from U.S. 12 east on 15th, to 20th Ave. then south on 20th.
   To existing 33, 34.

2. Traffic control on this project shall be in accordance with Standard Road Plans RS-1, RS-2, RS-3, RS-4A, RS-5A, RS-7, RS-8, RS-9, RS-10, RS-11, RS-12. For additional complimentary information, refer to Part VI of the Manual on Uniform Traffic Control Devices and the current Standard Specifications.

3. The contractor will coordinate traffic control with other projects in the area.

4. All traffic control devices shall be furnished, erected, maintained, and removed by the contractor.

5. Where possible, all uncontrolled signals shall be placed at least 20 ft. behind the curb or edge of
   shoulder.

6. The location for storage of equipment by the contractor during non-working hours shall be as
   approved by the engineer in charge of construction.

7. The engineer may require modifications to the pavement marking details shown. Conflicting
   permanent edge lines, center lines, or lane lines shall be removed. As permanent edge lines, center
   lines, and lane lines may be placed before the roadway is returned to normal traffic.

8. Proposed sign spacing may be modified, as approved by the engineer, to meet existing field
   conditions or to prevent obstruction of the neck view or permanent signing.

9. Permanent signing that conveys a message contrary to the message of the temporary signing and
   not applicable to the working conditions shall be covered by the contractor when directed by the
   engineer.

10. Proposed changes in the traffic control plan shall be reviewed for approval by the engineer
    before changes are made.
### STANDARD ROAD PLANS

<table>
<thead>
<tr>
<th>NUMBER</th>
<th>DATE</th>
<th>NUMBER</th>
<th>DATE</th>
<th>NUMBER</th>
<th>DATE</th>
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### BRIDGE APPROACH SECTION

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### TABULATION OF SILT FENCES

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<td>14-05-000.83</td>
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</tr>
<tr>
<td></td>
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</table>

**DESIGN NO. 203**

**FILE NO. 23907**
Appendix C
Moment Magnification Calculations

Moment Magnification Calculations for Load Combination 1010 for Bottom of Column 1
Aashto Lrfd 5.7.4.3 and 4.5.3.2.2b

Factored Load Reactions from RC-Pier

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y_1$ = 727.65 k</td>
<td>$F_y_2$ = 709.02 k</td>
<td>$F_y_3$ = 597.08 k</td>
</tr>
<tr>
<td>$M_{x1}$ = 313.24 k*ft</td>
<td>$M_{x2}$ = 311.86 k*ft</td>
<td>$M_{x3}$ = 313.24 k*ft</td>
</tr>
<tr>
<td>$M_{z1}$ = -25.43 k*ft</td>
<td>$M_{z2}$ = -11.38 k*ft</td>
<td>$M_{z3}$ = 23.60 k*ft</td>
</tr>
</tbody>
</table>

RC-Pier assumes minimum eccentricity according to Aashto Std. Spec. 8.16.5.2.8

$e_{min} = 0.6 + 0.03*h = 0.6 + (0.03)*(30") = 1.5" = 0.125'$

$M_{x_{min1}} = M_{x_{min2}} = (F_y_1)*(e_{min}) = (727.65 k)*(0.125') = 90.956 k*ft$

$M_{x_{min2}} = M_{x_{min3}} = (F_y_2)*(e_{min}) = (709.02 k)*(0.125') = 88.628 k*ft$

$M_{x_{min3}} = M_{x_{min3}} = (F_y_3)*(e_{min}) = (597.08 k)*(0.125') = 74.635 k*ft$

Factored Loads Considered

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y_1$ = 727.65 k</td>
<td>$F_y_2$ = 709.02 k</td>
<td>$F_y_3$ = 597.08 k</td>
</tr>
<tr>
<td>$M_{x1}$ = 313.24 k*ft</td>
<td>$M_{x2}$ = 311.86 k*ft</td>
<td>$M_{x3}$ = 313.24 k*ft</td>
</tr>
<tr>
<td>$M_{z1}$ = -90.956 k*ft</td>
<td>$M_{z2}$ = -88.628 k*ft</td>
<td>$M_{z3}$ = 74.635 k*ft</td>
</tr>
</tbody>
</table>

Moment Magnification from Aashto Lrfd 4.5.3.2.2b with RC-Pier Modifications

$M_c = \delta_b M_{2b} + \delta_s M_2$

RC-Pier modifies this equation for unbraced frames by assuming that all moments are to be magnified by $\delta_s$ alone.

where $\delta_s = 1 / [1 - \Sigma P_u/(\phi_k*\Sigma P_c)]$

$\phi_k = 0.75$  Stiffness reduction factor for concrete

$P_c = \pi^2*EI/(k*l_u)^2$  Euler buckling load

$EI = (E_c*I_g/2.5)/(1 + d)$  Flexural column stiffness

$\beta_d$ is ratio of maximum factored dead load moment to maximum factored total moment, always positive

Calculate $\beta_d = |\text{Maximum Factored Dead Load Moment} / \text{Maximum Factored Total Load Moment}|$

Loads from RC-Pier

Unfactored Self-weight

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y_1$ = 40.10 k</td>
<td>$F_y_2$ = 44.88 k</td>
<td>$F_y_3$ = 40.10 k</td>
</tr>
<tr>
<td>$M_{x1}$ = 0.00 k*ft</td>
<td>$M_{x2}$ = 0.00 k*ft</td>
<td>$M_{x3}$ = 0.00 k*ft</td>
</tr>
<tr>
<td>$M_{z1}$ = -0.41 k*ft</td>
<td>$M_{z2}$ = 0.00 k*ft</td>
<td>$M_{z3}$ = 0.41 k*ft</td>
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</tbody>
</table>

Unfactored DC loads

<table>
<thead>
<tr>
<th>Column 1</th>
<th>Column 2</th>
<th>Column 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y_1$ = 305.22 k</td>
<td>$F_y_2$ = 282.22 k</td>
<td>$F_y_3$ = 305.22 k</td>
</tr>
<tr>
<td>$M_{x1}$ = 0.00 k*ft</td>
<td>$M_{x2}$ = 0.00 k*ft</td>
<td>$M_{x3}$ = 0.00 k*ft</td>
</tr>
<tr>
<td>$M_{z1}$ = 4.87 k*ft</td>
<td>$M_{z2}$ = 0.00 k*ft</td>
<td>$M_{z3}$ = -4.87 k*ft</td>
</tr>
</tbody>
</table>
Factored Self-weight (Load factor = 1.25)

\[ \begin{align*}
F_y1 &= 50.125 \text{k} \\
F_y2 &= 56.10 \text{k} \\
F_y3 &= 50.125 \text{k} \\
M_x1 &= 0.00 \text{k*ft} \\
M_x2 &= 0.00 \text{k*ft} \\
M_x3 &= 0.00 \text{k*ft} \\
M_z1 &= -0.5125 \text{k*ft} \\
M_z2 &= 0.00 \text{k*ft} \\
M_z3 &= 0.5125 \text{k*ft}
\end{align*} \]

Factored DC loads (Load factor = 1.25)

\[ \begin{align*}
F_y1 &= 381.525 \text{k} \\
F_y2 &= 352.775 \text{k} \\
F_y3 &= 381.525 \text{k} \\
M_x1 &= 0.00 \text{k*ft} \\
M_x2 &= 0.00 \text{k*ft} \\
M_x3 &= 0.00 \text{k*ft} \\
M_z1 &= 6.0875 \text{k*ft} \\
M_z2 &= 0.00 \text{k*ft} \\
M_z3 &= -6.0875 \text{k*ft}
\end{align*} \]

Factored Self-weight + DC loads

\[ \begin{align*}
F_y1 &= 431.650 \text{k} \\
F_y2 &= 408.875 \text{k} \\
F_y3 &= 431.650 \text{k} \\
M_x1 &= 0.00 \text{k*ft} \\
M_x2 &= 0.00 \text{k*ft} \\
M_x3 &= 0.00 \text{k*ft} \\
M_z1 &= 5.575 \text{k*ft} \\
M_z2 &= 0.00 \text{k*ft} \\
M_z3 &= -5.575 \text{k*ft}
\end{align*} \]

Check \( M_{\text{min}} \) due to minimum eccentricity (\( e_{\text{min}} = 0.125' \))

\[ \begin{align*}
M_{x\text{min}1} &= M_{z\text{min}1} = (431.650 \text{k})*(0.125') = 53.956 \text{k*ft} \\
M_{x\text{min}2} &= M_{z\text{min}2} = (408.875 \text{k})*(0.125') = 51.109 \text{k*ft} \\
M_{x\text{min}3} &= M_{z\text{min}3} = (431.650 \text{k})*(0.125') = 53.956 \text{k*ft}
\end{align*} \]

Factored Loads considered for \( \beta_d \)

\[ \begin{align*}
F_y1 &= 431.650 \text{k} \\
F_y2 &= 408.875 \text{k} \\
F_y3 &= 431.650 \text{k} \\
M_x1 &= 53.956 \text{k*ft} \\
M_x2 &= 51.109 \text{k*ft} \\
M_x3 &= 53.956 \text{k*ft} \\
M_z1 &= 53.956 \text{k*ft} \\
M_z2 &= 51.109 \text{k*ft} \\
M_z3 &= -53.956 \text{k*ft}
\end{align*} \]

\( \beta_d \) Calculations

\[ \begin{align*}
\beta_{dx1} &= \left| \frac{(53.956 \text{k*ft})}{(313.24 \text{k*ft})} \right| = 0.172252 \\
\beta_{dz1} &= \left| \frac{(53.956 \text{k*ft})}{(-90.956 \text{k*ft})} \right| = 0.593211 \\
\beta_{dx2} &= \left| \frac{(51.109 \text{k*ft})}{(311.86 \text{k*ft})} \right| = 0.163886 \\
\beta_{dz2} &= \left| \frac{(51.109 \text{k*ft})}{(-88.628 \text{k*ft})} \right| = 0.576676 \\
\beta_{dx3} &= \left| \frac{(53.956 \text{k*ft})}{(313.24 \text{k*ft})} \right| = 0.172252 \\
\beta_{dz3} &= \left| \frac{(53.956 \text{k*ft})}{(-74.635 \text{k*ft})} \right| = 0.722935
\end{align*} \]

Calculate \( EI = \left( E_c I_g / 2.5 \right) / (1 + \beta_d) \)

\[ \begin{align*}
E_c &= (33)*(150 \text{pcf})^{1.5}* (3500 \text{ psi})^{0.5} * [(144 \text{ in}^2/\text{ft}^2) / (1000 \text{ lb/k})] = 516,472.7 \text{ ksf} \\
I_g &= 0.25*\pi*6^{4} = 0.25*\pi*(0.5*2.5')^{4} = 1.9175 \text{ ft}^4
\end{align*} \]

\[ \begin{align*}
\text{Column 1} & \quad EI_{x1} = [(516,472.7 \text{ ksf})*(1.9175 \text{ ft}^4) / 2.5] / (1 + 0.172252) = 337,921.8 \text{ k*ft}^2 \\
& \quad EI_{z1} = [(516,472.7 \text{ ksf})*(1.9175 \text{ ft}^4) / 2.5] / (1 + 0.593211) = 248,636.0 \text{ k*ft}^2 \\
\text{Column 2} & \quad EI_{x2} = [(516,472.7 \text{ ksf})*(1.9175 \text{ ft}^4) / 2.5] / (1 + 0.163886) = 340,350.9 \text{ k*ft}^2 \\
& \quad EI_{z2} = [(516,472.7 \text{ ksf})*(1.9175 \text{ ft}^4) / 2.5] / (1 + 0.576676) = 251,243.4 \text{ k*ft}^2 \\
\text{Column 3} & \quad EI_{x3} = [(516,472.7 \text{ ksf})*(1.9175 \text{ ft}^4) / 2.5] / (1 + 0.172252) = 337,921.8 \text{ k*ft}^2 \\
& \quad EI_{z3} = [(516,472.7 \text{ ksf})*(1.9175 \text{ ft}^4) / 2.5] / (1 + 0.722935) = 229,915.6 \text{ k*ft}^2
Calculate \( P_e = \frac{(\pi^2*EI)}{(k*l_u)^2} \)

\( k_x = 2.1, k_z = 1.2, l_u = 20.5' \)

**Column 1**

\[
P_{ex1} = \frac{(\pi^2)*(337,921.8 \text{ k*ft}^2)}{[(2.1)*(20.5')^2]} = 1799.574 \text{ k}
\]

\[
P_{ez1} = \frac{(\pi^2)*(248,636.0 \text{ k*ft}^2)}{[(1.2)*(20.5')^2]} = 4055.025 \text{ k}
\]

**Column 2**

\[
P_{ex2} = \frac{(\pi^2)*(340,350.9 \text{ k*ft}^2)}{[(2.1)*(20.5')^2]} = 1812.51 \text{ k}
\]

\[
P_{ez2} = \frac{(\pi^2)*(251,243.4 \text{ k*ft}^2)}{[(1.2)*(20.5')^2]} = 4097.55 \text{ k}
\]

**Column 3**

\[
P_{ex3} = \frac{(\pi^2)*(337,921.8 \text{ k*ft}^2)}{[(2.1)*(20.5')^2]} = 1799.574 \text{ k}
\]

\[
P_{ez3} = \frac{(\pi^2)*(229,915.6 \text{ k*ft}^2)}{[(1.2)*(20.5')^2]} = 3749.712 \text{ k}
\]

Calculate \( \delta_x = \frac{1}{1 - \frac{\Sigma P_u}{(\phi \Sigma P_e)}} \) for Column 1

**Column 1**

\[
\delta_{sx} = \frac{1}{1 - \left(\frac{727.65 \text{ k} + 709.02 \text{ k} + 597.08 \text{ k}}{(0.75)*(1799.574 \text{ k} + 1812.51 \text{ k} + 1799.574 \text{ k})}\right)}
= 2.0043
\]

\[
\delta_{sz} = \frac{1}{1 - \left(\frac{727.65 \text{ k} + 709.02 \text{ k} + 597.08 \text{ k}}{(0.75)*(4055.025 \text{ k} + 4097.55 \text{ k} + 3749.712 \text{ k})}\right)}
= 1.2950
\]

**Factored Loads with Magnification for Column 1**

\( F_{y1} = 727.65 \text{ k} \)

\( M_{x1} = (313.24 \text{ k*ft})*(2.0043) = 627.827 \text{ k*ft} \)

\( M_{z1} = (-90.956 \text{ k*ft})*(1.2950) = -117.793 \text{ k*ft} \)
Appendix D
RC-Pier and Footing Surcharge

The user needs to be aware of how RC-Pier handles footing surcharge loads:

- Load factors are not applied to the footing surcharge if the EV load case is not specified on the Loads tab.

The problem will be illustrated using a simple example. Consider the unusual pier below. The cap is 2’ x 2’ x 2’. The column is 2’ x 2’ x 8’ tall. The footing is 10’ x 10’ x 0.5” thick. There is one pile in the center of the footing. Two bearings are centered over the column.
I modeled the pier like this to make it easier to see what the loads are. Essentially the footing weight is 0 kips because it is very thin and because I set the unit weight of the footing to the really small value of 0.101 pcf (see below).

On the Loads tab I selected only load type DC; however, I didn’t enter any loads for the DC1 load type. Notice that I also selected Strength Group 1.
The only load on my pier footing at this point is the self-weight of the cap and column. Remember that the footing is “weightless”.

Unfactored Cap + Column Weight = (2’)*(2’)*(2’ + 8’)*(0.150 kcf) = 6 kips
Max. Factored Cap + Column Weight = (1.25)*(6 kips) = 7.50 kips
Min. Factored Cap + Column Weight = (0.90)*(6 kips) = 5.40 kips

If I check the Design Status of my footing pile (since I only have one pile) in RC-Pier I can see that my calculations above concur with RC-Pier’s results for the maximum and minimum factored pile reaction.

Now let’s add in a footing surcharge load and see how RC-Pier handles that. We will assume we have 2’ of fill on the footing and that the unit soil weight is 0.120 kcf.

Constant Footing Surcharge = (2’)*(0.120 kcf) = 0.240 ksf
When we check the Design Status of our footing in RC-Pier we get the following results:

The maximum factored pile reaction is now 31.50 kips. We can calculate this as follows:

\[
\text{Unfactored Surcharge Load} = (0.240 \text{ ksf}) \times (10') \times (10') = 24.00 \text{ kips}
\]
\[
\text{Max. Pile Rxn} = (\text{Max. Factored Cap} + \text{Column Weight}) + (\text{Unfactored Surcharge Load})
\]
\[
\text{Max. Pile Rxn} = 7.50 \text{ kips} + 24.00 \text{ kips} = 31.50 \text{ kips}
\]

So we can see that the surcharge load is not being factored. Now let’s see what happens when we add the EV load on the Loads tab screen as shown below. Note that we are not actually entering any load as an EV1 load or as a DC1 load.
This time we get the different results as shown below.

<table>
<thead>
<tr>
<th>Pile Reactions, Factored</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pile</strong></td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

**Footing Design : Notes**

Only max. force in piles is considered for design.

Pile coordinates X and Z are from the most left edge of the footing.

Plong= Lateral load in longitudinal direction at the top of pile, kips.

Ph= Available resisting horizontal component due to batter= batter * Vertical pile reaction, kips.

Plong-Ph= Remaining lateral force required to resist by pile.

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

Factored pile reaction 7.50 kips

The maximum factored pile reaction is now 39.90 kips. We can calculate this as follows:

Factored Surcharge Load = \((1.35) \times (24.00 \text{ kips})\) = 32.40 kips

Max. Pile Rxn = (Max. Factored Cap + Column Weight) + (Factored Surcharge Load)

Max. Pile Rxn = 7.50 kips + 32.40 kips = 39.90 kips

So we can see that the surcharge load is being factored when we add EV to the Loads tab screen. [Note that I also tried an ES load instead of an EV load. The ES load did not result in a load factor being applied to the Footing Surcharge load.] So, the Footing Surcharge is only factored when the EV load is specified on the Loads tab screen. Excluding the load factor for the fill loads on the footing can be fairly significant if you have a deep fill and relatively light superstructure loads.

So, if you typically enter the footing surcharge load on the Footing tab then you should still supply an EV load on the Loads tab even if you don’t enter a load for it. Apparently RC-Pier hasn’t always functioned in this manner. An office example I put together for 305 Wapello around 10/10/2006 (RC-Pier Version 4.1.0) shows some calculations that make it apparent that the load factor was being included in the footing surcharge load when it was entered on the Footing tab, but no EV load was specified on the Loads tab.

One recommended procedure might be to enter the footing fill load for each pier footing as an EV load near the bottom of each column on the Loads tab. [I recommend the load be placed just a fraction above the bottom of the column for the RC-Pier footing design runs since that ensures the Analysis Results on the Analysis tab reflect the load.] Doing it this way may affect whether or not you want to make use of RC-Pier to design your footing reinforcement since it will have some effect on those results.
An interesting side note concerns the footing self-weight applied by RC-Pier. Not that you would ever do this, but… if you were ever to do a run of RC-Pier with no DC loads then you would find that the cap and column self-weight would not be included as a load on your footing. However, the footing self-weight would be applied to your footing, but it would not have a load factor applied.

Finally it should be noted that the column area is not deducted from the footing area when the footing surcharge is actually computed. Normally this isn’t a big deal since the column area is generally quite a bit smaller than the footing area, but it is something to keep in mind.
Appendix E
As you may have seen, RC-Pier allows the user to enter pile batter in the z-axis direction only. For instance, in the figure below I have entered a batter of 14.03 degrees which is approximately a 1:4 batter. When a pile batter is entered the program prints some additional output to the Pile Reactions table. Some of the calculations for this additional output are demonstrated on the following pages.
For these example calculations we are going to run only one combination (#79).

\[
\text{Comb } \# \quad 79 \quad \text{(IA STA 1)} \quad = \quad 1.00 \quad (\quad 1.25 \text{ D}C1 + 1.50 \text{ D}W1 + 1.35 \text{ I}V1 + 1.75 \text{ L}L1 \\
- \quad 1.75 \text{ \text{B}E1} + 0.50 \text{ TJ1} + 0.50 \text{ \text{SH}Z} \quad )
\]

We are only going to look at the calculations for footing 1 which corresponds with node 1 of member 1. The forces for combination 79 at node 1 are as follows.

The pile reactions for this footing are as follows. The last three columns of output are printed because we have input a pile batter.
The following calculations are developed for pile #1.
The section moduli for pile #1 in the X and Z directions with respect to the center of the footing are:

\[ S_x = \frac{(2 \text{ rows})*(3 \text{ piles})*(3')^2}{3'} = 18 \text{ pile*ft} \]
\[ S_z = \frac{(2 \text{ rows})*(2 \text{ piles})*(3')^2}{3'} = 12 \text{ pile*ft} \]

The pile reaction for pile #1 is:

\[ P_{\text{Rxn}} = \left[ \frac{880.62 \text{ k}}{7 \text{ piles}} \right] + \left[ \frac{253.16 \text{ k*ft}}{18 \text{ pile*ft}} \right] - \left[ \frac{121.55 \text{ k*ft}}{12 \text{ pile*ft}} \right] = 129.74 \text{ k} \]

The horizontal component of the pile reaction due to pile batter is:

\[ P_{\text{h}} = (129.74 \text{ k}) \times (\tan(14.03 \text{ deg})) = 32.42 \text{ k} \]

The lateral forces on the pile are as follows. RC-Pier currently ignores Fx forces since they are perpendicular to the direction of batter.

\[ F_x = \left( \frac{12.31 \text{ k}}{7 \text{ piles}} \right) = 1.76 \text{ k} \]
\[ F_z = \left( \frac{-9.941 \text{ k}}{7 \text{ piles}} \right) = -1.42 \text{ k} \]

\[ \text{Plong} = F_z = -1.42 \text{ k} \]
The sign convention for this lateral force and the horizontal component of the pile reaction are opposed and thus RC-Pier assumes:

$$\text{Plong-Php} = 32.42 \text{ k} - 1.42 \text{ k} = 31.00 \text{ k}$$

It appears (from additional testing) that RC-Pier flags any positive value for “Plong-Php” as a failure. The additional RC-Pier output for battered piles is somewhat confusing and, for the time being, you should simply refer to the Bridge Design Manual for guidance in dealing with lateral pile forces for vertical and battered piles.

BDM 6.6.4.1.3.1
“The pile group supporting a pier footing shall be checked for lateral loading [OBS MM No. 9]. Each vertical pile may be assumed to have shear resistance, and each battered pile may be assumed to have shear resistance plus the horizontal component of the axial resistance. See the pile resistance guidelines for steel H-piles [BDM 6.2.6.1] and for timber piles [BDM 6.2.6.3].”
Appendix F
This run will be used to illustrate footing design in RC-Pier. I’m only going to show screens that have been modified from the cap/column run.

The columns have been extended 3.5’ to the bottom of the footing for this footing design run.

In RC-Pier, the footing and pile are not part of the structural model.

The footing and fill weight will be placed using the entries on the “Footing” tab since I am interested in looking at RC-Pier’s methodology for footing design.

Pile batter was not included.
Notice that the column length has been increased by 3.5’ from 22’ to 25.5’.
The footing concrete density was left as 150 pcf since the self-weight will be calculated by RC-Pier.

I removed all the additional pier cap check points.
I added an EV1 load for the fill weight on the footing. Even though no load will be entered here this will ensure the load factor is applied to the footing surcharge as entered on the footing tab.
Removed the weight due to the 3.5’ column extension to counteract the addition of its self-weight: 

\[ \frac{\pi}{4}(0.5 \times 2.5^2) \times (3.5) \times (0.150 \text{ kcf}) = 2.577 \text{ k} \]

The reduction in column weight is placed just above the bottom of the column to ensure it is included in the analysis results.

I didn’t adjust the locations of the wind loads since the overall effect is not that significant. However, if I had the new values would be:

Start: \( y_1/L = 7.5' / 24' = 0.313 \)

End: \( y_2/L = 21.5' / 24' = 0.896 \)
The fill weight is not entered here. It will be entered on the footing tab as a surcharge. This blank entry is needed so that RC-Pier applies the EV load factor to the surcharge on the footing tab.
All dynamic load allowance factors were set to 0 to ensure that the analysis results exclude impact if they are written to a file.
Footing thickness was set to 3.5’.

The fill weight is entered as a surcharge:

\[(4’ \text{ fill}) \times (0.120 \text{ kcf}) = 0.480 \text{ ksf}\]
This information is from a library – see following pages.

This is used for graphics display.

Not used in the structural model – it is simply used in the graphic display. I keep the pile short so they don’t take up the whole picture.

This value is arbitrary since the Iowa DOT currently bases pile design on the Strength and Extreme Event Combinations.

HP10x57 Structural Resistance Level 1

Factored Resistance = (6 ksi)*(0.1167 ft²)*(144 in²/ft²)*(1.45) = 146.16 k

BDM Table 6.2.6.1-1 shows (0.6)*(243 k) = 145.8 k
Enter pile coordinates. Pile batter was not included.
Footing reinforcement was entered according to the plans.

This button lets you review the design.
I've included some portions of RC-Pier's output for the footing.

Not interested in the Service capacity of the piles at this time.

Maximum Service Reaction
This is greater than the factored resistance of 145.80 k. So, I should modify my pile arrangement or add more piling. I won’t do that at this time.

Note that the maximum factored pile reaction is not 154.04 kips in this table. This is because the footing and surcharge (fill weight) have been deducted. The same is true for the service pile reaction.

Footing Self-weight = (0.150 kcf)*(9’)*(9’)*(3.5’) / (7 piles) = 6.075 k/pile
Surcharge = (0.120 kcf)*(9’)*(9’)*(4’ fill depth) / (7 piles) = 5.5543 k/pile
Factored Pile Reaction = 154.04 k – (1.25)*(6.075 k) – (1.35)*(5.5543 k) = 138.95 k
Service Pile Reaction = 125.24 k – (1.00)*(6.075 k) – (1.00)*(5.5543 k) = 113.61 k
Note that the column footprint in the fill is not deducted.
As = 0.0015*Ag/2 = (0.0015)*(9')*(12 in/ft)*(3.5')*(12 in/ft) / 2 = 3.402 in²

This appears to be based on the 2005 Aashto Lrfd Code Art. 5.10.8.2

Asb required

Flexure

Flexure Note

CL: Section classification as per LRFD 2006 interns for provided reinforcement.
C = Compression controlled, I = In-Transition, T = Tension controlled.
Required reinforcement is based on phi for tension controlled sections...
* The provided reinforcement is not adequate, either less than required or larger than maximum allowed.

Cracking check as per AASHTO LRFD 2007 with Interims (2009)

Cracking/Fatigue

Cracking/Fatigue Note

* Provided relar spacing is not adequate for crack control.

*** Spacing is negative.

One Way Shear (Simplified Method)

See hand calculations for one-way shear

Two Way Shear

See hand calculations for two-way shear

Two Way Shear Note

TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.
RC-Pier Footing Design

Equivalent Square Column

\[ W = \sqrt{11 \left(\frac{2.5'}{2}\right)^2} = 2.216' \]

\[ W = 1.108' \]

Flexure

X-Dir : 7-#9s

\[ d_s = 42'' - 13'' = 28.428'' \]

\[ M_{max} = (138.95 ksi/pile)(2 piles)(3' - 1.108') = 525.8 \text{ k-in} \]

\[ A_{s, prov} = 7bars \times 1.00in^2 = 7in^2 \]

\[ \alpha = \frac{A_s f_y}{0.85 \varepsilon_b} = \frac{(7in^2)(60ksi)}{(0.85)(3.5ksi)(9'x12')} = 1.307'' \]

\[ M_r = \phi A_s f_y \left( d_s - \frac{a}{2} \right) = (0.9)(7in^2)(60ksi)(28.428' - \frac{1.307''}{2}) = 10,30f \text{ k-in} \]

\[ = 875.15 \text{ k-in} \]

Check if Tension-Controlled?

Yes

\[ \beta_1 = 0.85 \]

\[ c = 0.65 \]

\[ \varepsilon_s = \frac{(28.428'' - 1.538'')(0.003)}{1.538''} = 0.02 > 0.005 \]

\[ \varepsilon_u = 0.003 \]

\[ \varepsilon_s = \frac{(d_s - c) \varepsilon_u}{c} \]
Find As req'd

\[ R_n = \frac{M_0}{\phi b ds} = \frac{(525.8^{k_1})(12^{\%})}{(0.9)(9^{\%} \times 12^{\%})(28.436^{\%})^2} = 0.08028 \text{ ksi} \]

\[ \phi = \frac{0.85 \phi_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85 \phi_c}}\right) = \frac{(0.85)(3.5 \text{ ksi})}{60 \text{ ksi}} \left(1 - \sqrt{1 - \frac{(2)(0.08028 \text{ ksi})}{(0.85)(3.5 \text{ ksi})}}\right) = 0.0013565 \]

As req'd = \phi b ds = (0.0013565)(9^{\%} \times 12^{\%})(28.436^{\%}) = 4.166 \text{ in}^2

\[ \frac{4}{3} \text{ As req'd} = (\frac{4}{3})(4.166 \text{ in}^2) = 5.555 \text{ in}^2 \quad \text{or} \quad \text{Matches RC-Pile} \]

\[ I_0 = \frac{1}{12} bh^3 = \frac{(12)(9^{\%} \times 12^{\%})(42^{\%})^3}{12} = 666,972 \text{ in}^4 \]

\[ S_f = 0.37 \sqrt{f_c} = (0.37)\sqrt{3.5 \text{ ksi}} = 0.6922 \text{ ksi} \]

\[ M_{cr} = \frac{S_f I_0}{c} = \frac{(0.6922 \text{ ksi})(666,972 \text{ in}^4)}{42^{\%}} = 21,979^{k_2} = 1831.6^{k_1} \]

\[ 1.2 M_{cr} = (1.2)(1831.6^{k_1}) = 2197.9^{k_1} \quad \text{>} \quad \frac{4}{3} M_0 = (\frac{4}{3})(525.8^{k_1}) = 701.1^{k_1} \]

\[ S_0 \text{ As req'd} = 5.555 \text{ in}^2 < \text{As prin} = 7 \text{ in}^2 \]

**Cracking**

Ashto Ltd 5.7.1.4

\[ S \leq \frac{700 \phi_c}{P_{cr}^{\phi_c}} - 2d_c \quad \text{< Look at bars in x-dir> } \]

\[ \phi_c = 1.00 \]

\[ d_c = 12^{\%} + 1^{\%} + \frac{1128^{\%}}{2} = 13.384^{\%} \]

\[ \beta_s = 1 + \frac{d_c}{0.7(h-d_c)} = 1 + \frac{13.384^{\%}}{(0.7)(42^{\%} - 13.384^{\%})} = 1.6814 \]

\[ S_{ss} \text{ is tensile stress in the R/I at Service limit state} \]

Max service pile reaction \( w = \text{footing} \times \text{friction} \)

\[ M_{max} = (113.61^{\%} / \text{piles})(2 \text{piles})(3'-1.108') = 429.90^{k_1} \]

\[ A_s = (7 \text{ bars})(1.0 \text{ in}^2) = 7.0 \text{ in}^2 \]

\[ d_s = 42^{\%} - 13^{\%} - \frac{1128^{\%}}{2} = 28.436^{\%} \]

\[ \phi = \frac{A_s}{Bds} = \frac{7 \text{ in}^2}{(9^{\%} \times 12^{\%})(28.436^{\%})} = 0.002779 \]

\[ 2\phi n = 0.036969 \]

\[ \frac{K}{E_c} = \frac{2\phi n}{K} = 8.086 \rightarrow 8 \]

\[ n = \frac{E_s}{E_c} = 0.086 \rightarrow 8 \]

\[ K = \sqrt{(2\phi n)^2 + 4\phi n} - 2\phi n = 0.236053 \]
j = 1 - \frac{k_s}{1} = 1 - \frac{0.236053}{3} = 0.9213

S_{ss} = \frac{M}{A_s \cdot d_s} = \frac{(429.90(12\%)}{(7.0 in^2)(0.9213)(28.436)} = 28.130 ksf \quad \text{close to RC-Pier}

S = \frac{(700)(1.00)}{(1.6814)(28.130 ksf)} - (2)(13.564") = -12.328" \quad \text{negative spacing}

\text{One Way Shear} \quad \text{Aashto Lfd 5.13.3.6 & 5.8.1.4} \quad 5.8.3.2

x-dir

\begin{align*}
d_v &= \text{max of} \begin{cases} 
0.72 h = (0.72)(42" = 30.24" \quad \leftarrow \text{controls} \\
0.9 d_s = (0.90)(28.436") = 25.5924" \\
d_s - \frac{h}{2} = 25.5924" - \frac{1.307"}{2} = 24.939" 
\end{cases}
\end{align*}

\text{Critical section} = 1.108' + \frac{30.24"}{12\%} = 3.628'

\text{All piles are inside critical section, shear design is not required.}

V_c = 0.0316 f_b v u \leq 0.28 f_c v u

\text{Let } f_b = 2.0

V_c = (0.0316)(2.0)(\sqrt{3.5 ksi})(9\times 12\%)(30.24")

= 386.15 k \quad < (0.28)(3.5 ksi)(9\times 12\%)(30.24") = 2857.68 k

\phi V_c = (0.9)(386.15 k) = 347.54 k \quad \leftarrow \text{matches RC-Pier}

\text{The piles in the x-dir will also be inside the critical section}
Two Way Shear  

\[ d_V = 30.24'' \text{ (based on 0.72h)} \]

Critical section = \(1.108 + \frac{30.24''}{(2)(12\%)}\) = 2.368''

(6 piles) (138.95 ksi/pile) = 833.7 k

\[ V_n = (0.063 + \frac{0.126}{B_c}) \sqrt{f'_c} b_0 d_V \leq \frac{0.126 \sqrt{f'_c}}{B_c} b_0 d_V \]

\[ B_c = \frac{(2)(2.368')}{(2)(2.368')} = 1 \text{ square} \]

\[ b_0 = (8)(2.368') = 18.944' = 227.33'' \]

\[ V_n = (0.063 + \frac{0.126}{1}) \sqrt{13.5 ksi} \times (227.33') \times (30.24'') = 2430.7 k \]

\[ \phi V_n - \phi V_c = (0.9)(1620.5 k) = 1458.4 k \]

\[ (0.126)(13.5 ksi)(227.33')(30.24'') \]

Not Used by RC-Piers

Columns  

\[ d_V = 30.24'' \]

Critical section = \(2\times1' + \frac{30.24''}{(2)(12\%)} = 2.51'' \)

(6 piles) (138.95 ksi/pile) = 833.7 k

\[ B_c = 1 \text{ circular} \]

\[ b_0 = 2 \pi R = (2)(\pi)(2.510') = 15.771' \]

\[ = 189.25'' \]

\[ V_n = (0.063 + \frac{0.126}{1}) \sqrt{13.5 ksi} (189.25'')(30.24'') = 2023.5 k \]

\( \phi V_n - \phi V_c = (0.9)(1349.03 k) = 1214.1 k \)

\[ 833.7 k \quad \text{OK} \]
**Interior Pile**

\[ d_v = 30.24'' \]

\[ b_0 = (2)(40.23'' + 40.46'') = 161.38'' = 13.448' \]

\[ B_c = \frac{40.46''}{40.23''} = 1.006 \]

\[ V_n = (0.063 + \frac{0.126}{1.006})(\sqrt{3.5 \text{ ksi}})(161.38'')(30.24'') \]

\[ = 1719.01 \text{ k} \quad (0.126)(\sqrt{3.5 \text{ ksi}})(161.38'')(30.24'') = 1150.37 \text{ k} \]

\[ \varnothing V_n = \varnothing V_c = (0.9)(1150.37 \text{ k}) = 1035.33 \text{ k} \quad \text{Close to RC-Pier, RC-Pier may be using 10'' for both sides of the pile} \]

**Corner Pile**

\[ d_v = 30.24'' \]

\[ b_0 = (2)(18'') + \frac{40.46'' + 40.23''}{2} = 76.345'' = 6.362' \]

\[ V_c = 0.126 \sqrt{f_c} b_0 d_v \]

\[ = (0.126)(\sqrt{3.5 \text{ ksi}})(76.345'')(30.24'') \]

\[ = 544.21 \text{ k} \]

\[ \varnothing V_c = (0.9)(544.21 \text{ k}) = 489.8 \text{ k} \quad \text{Close to RC-Pier} \]
Appendix H
Hand Calculations for
Pile Footing Design Spreadsheet

$R_r = \varnothing R_n = 145.8^k$  \textit{Factored Pile Resistance used to design for one-way beam shear.}

$P_{um} = 153.886^k$  \textit{Maximum factored pile load used to design flexure R/S.}

$P_{ua} = 131.374^k$  \textit{Maximum factored average pile load used to design for two-way punching shear.}

\textbf{Note:} Normally $P_{um}$ should be less than $R_r$. However, the point of these calculations is to simply demonstrate the procedure.

The footing weight will not be deducted for the design of the flexural reinforcement and shear capacity of the footing.

\[ w = \sqrt{\pi \left( \frac{2.5}{2} \right)^2} = 2.216' \]

\[ \frac{w}{2} = 1.108' \]
Flexure

X - Dir

\[ ds = 42" - 13" - \frac{1.125"}{2} = 28.436" \]

\[ A_s = (7)(1.00in^2) = 7.0 \text{ in}^2 \]

\[ M_{u_x} = (153.586K)(2\text{plcs})(3' - 1.108') = 581.2\text{ k-in} \]

\[ a = \frac{A_s f_y}{0.85 f_{ce}} = \frac{(7\text{in})(60\text{ksi})}{0.85(3.5\text{ksi})(9\text{ksi})} = 1.307" \]

\[ M_r = \phi A_s f_y (ds - \frac{a}{2}) = 0.9(7\text{in})(60\text{ksi})(28.436" - \frac{1.307"}{2}) = 10,501.8 \text{ k-in} \]

\[ = 875.1 \text{ k-ft} \]

Check if tension-controlled

\[ \beta_1 = 0.85 \]

\[ c = \frac{a}{\beta_1} = \frac{1.307"}{0.85} = 1.538" \]

\[ \epsilon_s = \frac{(28.436" - 1.538")}{1.938"} (0.003) \]

\[ = 0.0525 > 0.005 \text{ : It is tension-controlled} \]

Check minimum reinforcement

Well assume we can use \( h = ds + 2" = 28.436" + 2" = 30.436" \)

\[ f_t = 0.37\sqrt{f'c} = (0.37)\sqrt{3.5\text{ksi}} = 0.692 \text{ ksi} \]

\[ I_g = \frac{d}{12}bh^3 = \left(\frac{d}{2}\right)(9\times12\%)(30.436")^3 = 253,749.5 \text{ in}^4 \]

\[ M_{cr} = \frac{f_t I_g}{c} = \frac{(0.692\text{ ksi})(253,749.5 \text{ in}^4)}{(30.436")^2} = 11,542.0 \text{ k-in}^2 = 961.8 \text{ k-ft} \]

\[ 1.2M_{cr} = 11,542.0 \text{ k-ft} > M_r \text{ : Note: } 1.2M_{cr} > \frac{4}{3}M_u \]

\[ M_r > \frac{4}{3}M_u \]

\[ R_n = \frac{M_u}{\phi bd_x} = \frac{(581.2\text{ k-in})(12\%)}{(0.9)(9\times12\%)(28.436")^2} = 0.08874 \text{ ksi} \]

\[ \rho = \frac{0.85 f'c}{f_y} (1 - \sqrt{1 - \frac{2R_n}{0.85 f'c}}) = \frac{(0.85)(3.5\text{ksi})}{(60\text{ksi})} (1 - \sqrt{1 - \frac{(2)(0.08874\text{ksi})}{(0.85)(3.5\text{ksi})}}) \]

\[ = 0.00150169 \]

\[ A_{\text{reg'd}} = \rho bd = (0.00150169)(9\times12\%)(28.436") = 4.612 \text{ in}^2 \]

\[ \frac{4}{3}A_{\text{reg'd}} = \left(\frac{4}{3}\right)(4.612 \text{ in}^2) = 6.149 \text{ in}^2 > A_{\text{prov}} = 7 \text{ in}^2 \]
Cracking

\[ S \leq \frac{700Y_e}{\frac{2}{3}s_{s55}} - 2d_c \quad \therefore \text{look at bars in x-dir} \]

\[ Y_e = 1.00 \]

Assume \( d_c = 2" \) and let \( h = ds + 2" = 28.436" + 2" = 30.436" \)

\[ B_s = 1 + \frac{d_c}{0.7(h-d_c)} = 1 + \frac{2}{0.7(30.436" - 2")} = 1.1005 \]

\( s_{s55} \) is tensile stress in the R/F at Service Limit State

\[ N_{\text{max}} = (124.9 \text{ kip})(2 \text{ piles})(3' - 1.108') = 472.6 \text{ kip} \]

\[ A_s = 7 \text{ in}^2 \]

\[ ds = 28.436" \]

\[ \rho = \frac{A_s}{bd} = \frac{7 \text{ in}^2}{(9' \times 12'')(28.436")} = 0.002279 \]

\[ \rho n = 0.0182346 \]

\[ K = \sqrt{\left(\rho n\right)^2 + 2\rho n - \rho n} = 0.1736 \]

\[ \beta = 1 - \frac{K}{2} = 1 - \frac{0.1736}{2} = 0.9421 \]

\[ s_{s55} = \frac{M}{A_sds} = \frac{(472.6 \text{ kip}')(12'')}{(7.0 \text{ in})(0.9421)(28.436"')} = 30.24 \text{ ksi} \]

\[ S \leq \frac{(700)(1.00)}{(1.1005)(30.24 \text{ ksi}) - (2)(2'')} = 17.034" \]
**One Way Shear**

\[ d_v = \max \{ \begin{cases} 0.72h = (0.72)(42') = 30.24' \quad \leftarrow \text{we are excluding this from consideration} \\ 0.9d_s = (0.9)(27.308') = 24.577' \\ d_s - \frac{a}{2} = 27.308' - 1.867' = 26.374' \quad \leftarrow \text{controls} \end{cases} \} \]

Critical section = 1.108' + \frac{26.374'}{12\%} = 3.306'

\[ \therefore \text{The 2 piles have a small portion outside the critical section, we may linearly interpolate to determine the shear load.} \]

\[ \frac{(3.4167' - 3.306') \cdot 12\%}{10'' \text{ pile}} = 0.133 = 13.3\% \]

\[ V_u = (145.8\%) \cdot (2 \text{ piles}) \cdot (0.133) = 38.78\text{k} \]

\[ V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b \cdot d_v < 0.25 \cdot f'_c \cdot b \cdot d_v \]
\[ = (0.0316)(2.0)(\sqrt{3.5\text{ksi}})(9' \times 12\%) \cdot (26.374') = 336.8\text{k} \]
\[ \therefore V_c = (0.9)(336.8\text{k}) = 303.1\text{k} > V_u \quad \therefore \text{OK} \]

\[ \therefore \text{To use } \beta = 2 \text{ the point of 0 shear to the Equiv. Column face must be less than } 3d_v \]

\[ 3.4167' - \frac{2.216'}{2} = 2.309' < 3d_v = (3)(\frac{26.374'}{12\%}) = 6.594' \quad \therefore \text{OK} \]
Two Way Shear

\[ d_v = 26.374" \]

Critical Section = \[ 1.108 + \frac{26.374"}{(2)(12\%)} \]
= \[ 2.207' \]

\( (6\text{piles})(131.374") = 788.2' \)

\[ V_c = (0.063 + \frac{0.126}{B_c}) \sqrt{f'_c \cdot b_o \cdot d_v} \leq \]
= \[ 0.126 \sqrt{f'_c \cdot b_o \cdot d_v} \]

\[ B_c = \frac{(2)(2.207')}{(2)(2.207')} = 1 \]

\[ b_o = (8)(2.207") = 17.664" = 211.872" \]

\[ V_c = (0.063 + \frac{0.126}{1}) \sqrt{3.5\text{ksi}} (211.872") (26.374") = 1975.8' \geq \]

\( (0.126)(\sqrt{3.5\text{ksi}})(211.872") (26.374") \)

\( \phi V_c = (0.9)(1317.2') = 1185.5' > V_o \)

\( \phi \) controls
Appendix I
CE – Vehicular Centrifugal Force

- CE is applied 6.0 feet above the deck surface to piers with horizontally curved roadways.
- Design speed for the appropriate highway classification shall be taken from the Office of Design’s Design Manual.
- Number of lanes loaded for CE shall be consistent with number of lanes loaded for vertical LL. Multiple presence factors apply to CE.
- Each pier shall resist the total CE force individually – it is not distributed among the bents.
The commentary of AASHTO LRFD 3.6.3 speaks of including and excluding CE in order to determine the worst case scenario for pier design. Our manual says CE should always be included when LL is included.

CE is based on a percentage of total truck (72 kips) or tandem (50 kips) axle weight, not a LL pier reaction of said weight. [The Iowa DOT does not consider 90% of two design trucks.]

\[
CE = C \times LL_{\text{truck or tandem}}
\]

\[
C = f \frac{v^2}{gR}
\]

where \(f = 4/3\)
Consider: 3 span PPC bridge (70.75’-91.5’-60.75’) -- This is not 305 Wapello
32 degree LA skew.
6 beam lines at 7.401’ (8.727’ skewed spacing).
Upstation Coordinate System.
Height of CE Above Cap = 6' + [8” Slab Thk + 54” Beam Hgt] / (12 in/ft) = 11.167’

C = f * v^2 / (g * R) = (4/3) * (102.67 ft/s)^2 / [(32.2 ft/s^2) * (500’)] = 0.8730

Design Truck Axle Weight = 32 k + 32 k + 8 k = 72 k

CE = Fx' = (72 k) * (0.8730) * (3 lanes) * (0.85) = 160.283 k  [MPF = 0.85]

Fx = -(160.283 k)*(cos(32 deg)) = -135.928 k

Fz = (160.283 k)*(sin(32 deg)) = 84.937 k

Fx per beam = (-135.928 k) / (6 beams) = -22.655 k

Fz per beam = (84.937 k) / (6 beams) = 14.156 k

Overturning Mom., Mz = (135.928 k)*(11.167’) = 1517.908 k*ft

Fy for beam 1 = -(1517.908 k*ft) / [(5 beam spa)*(8.727’ skewed)] = -34.786 k

Fy for beam 6 = (1517.908 k*ft) / [(5 beam spa)*(8.727’ skewed)] = 34.786 k

Overturning Mom., Mx = (84.937 k)*(11.167’) = 948.491 k*ft

Office policy is to delete Mx since we assume the connection between the pier and slab cannot transmit a moment in that direction.