This section contains guidance for the design and detailing of abutments, piers, retaining walls, and sheet pile walls. Abutments and piers are used to support bridge superstructures, whereas walls primarily function as earth retaining structures. In most cases, abutments, piers, and walls are reinforced concrete elements.

The preferred details for connecting the superstructure to the substructure are dependent on the geometry and type of bridge. For example, flexible substructure units supported by a single line of piles may be constructed integral with the superstructure. Conversely, short stiff substructure units are detailed with expansion bearings between the superstructure and substructure to reduce the design loads in the substructure units.

11.1 Abutments

General

Abutments function as both earth retaining structures and as vertical load carrying components. Parapet abutments are detailed to accommodate thermal movements with strip seal or modular expansion devices between the concrete deck and the abutment end block. Integral and semi-integral abutments are designed to accommodate movements at the roadway end of the approach panel.

Railroad bridge abutments shall be designed according to AREMA Specifications for the live load specified by the railroad. Design all other abutments according to the AASHTO LRFD Specifications. The Duluth Mesabe & Iron Range Railway requires a special live load. Construction will be in accordance with Mn/DOT Construction Specifications. The live load surcharge is found by taking the axle load and distributing it over an area equal to axle spacing multiplied by the track spacing, generally 70 square feet. Do not reduce the surcharge loading for skew.

Abutment Type Selection

Integral abutments are the preferred type of abutment when all of the following criteria are met:

- The bridge length and skew meet one of the following: (See Figure 11.1.1)
  - Bridge length ≤ 300 ft and skew ≤ 20 degrees
  - Bridge length ≤ 100 ft and skew ≤ 45 degrees
  - Bridge length is between 100 ft and 300 ft and skew ≤ 45 degrees - 0.125 (L - 100) where L is the length of the bridge in feet.
- Bridge horizontal alignment is straight. Slight curvature can be allowed, but must be considered on a case-by-case basis.
• The length of wingwall cantilevers are ≤ 14 feet (measured from the back face of abutment to the end of the wingwall).
• Abutment wingwalls do not tie into roadway retaining walls.
• Bridge configuration allows setting the abutment front face exposure on the low side of the bridge at 2 feet.
• Depth of beams is ≤ 72 inches.

![Figure 11.1.1](image)

**Figure 11.1.1**

Semi-integral abutments are the preferred type of abutment when the wingwall length, abutment exposure or superstructure depth requirements for integral abutments cannot be met. Semi-integral abutments must meet the bridge length, skew and horizontal alignment criteria listed above for integral abutments.

Parapet abutments should only be considered where integral and semi-integral abutment criteria cannot be met.

A parapet abutment behind a mechanically stabilized earth (MSE) retaining wall can be used where high abutments would be required and where it is economical to use an MSE wall. Locate the front face of the MSE wall a minimum of 6'-0" from the centerline of bearing. Do not batter the piles. Place the bottom of the abutment footing and the bottom of the MSE wall cap at the same elevation. Slope protection between the abutment and the MSE wall cap should not exceed a 1:4 slope.
Detailing/Reinforcement
For bridge rail sections that extend beyond the bridge ends and connect to guardrail, it is preferable to locate them on top of the approach panel rather than on top of the wingwall. However, for situations where the wingwalls tie into roadway retaining walls, be aware that this will result in an offset between the wingwall and roadway retaining wall. In this case, additional coordination with the roadway designer will be required.

Extend architectural rustications 2 feet below the top of finished ground.

As a minimum, tie abutment and wingwall dimensions to the working points by providing distances normal and parallel to the center line of bearing from working points to the following points:

- Centerline of piles at abutment footing corners
- Corners of abutment front face
- Corners of abutment fillets
- Wingwall ends at front or back face of wall

The gutter line, the edge of deck and the centerline of the fascia beam should be illustrated and labeled in the corner details.

To facilitate plan reading, label the ends of the abutments in the details (South End, North End, etc.).

Label all construction joints and identify the nominal size of keyways.

Where conduit extends through an abutment, provide horizontal dimensions from a working point to the location where the conduit penetrates the front face of the abutment or the outside face of the wingwall. The elevation at mid-height of the conduit should also be provided.

For presentation clarity, detail abutments with complicated layouts on separate sheets. Identical abutments (except for minor elevation differences) should be detailed on common sheets.

The minimum depth for the paving bracket is 1'-4".

On footing details, specify the lap splice length for bent dowels and the dowel projection for straight dowels.

If the railing contains a separate end post (supported on the abutment), show the end post anchorage reinforcement in the abutment details.
Membrane waterproofing (per Spec. 2481.3B) shall be provided for construction joints, doweled cork joints, Detail B801 joints, and on wall joints below ground. Waterproofing is not required at the top of parapet expansion block joint.

All reinforcement, except those completely in the footing, shall be epoxy coated. The minimum size of longitudinal reinforcement in abutment and wingwall footings is No. 19 bars.

On semi-integral and parapet abutments, provide pedestals under bearings and slope bridge seat for drainage between beams. Pedestals should be set back 2" from front face of abutment. Minimum pedestal height is to be 2" at front of pedestal. For bearing pedestals over 2 inches tall provide No. 13 or No. 16 reinforcing tie bars at 6 inch to 8 inch centers in both directions under the bearings. For pedestals with a height of 2 inches, only the reinforcement transverse to the abutment is required. Horizontal steel in pedestals should have 2 inches of clear cover to bridge seat. Provide a minimum of 2 inches of clear distance between anchor rods and reinforcing tie bars.

Figure 11.1.2 illustrates minimum cover and clearance dimensions for abutments.

![Minimum Cover and Clearance Figure 11.1.2](image-url)
Provide shrinkage and temperature reinforcement per Article 5.2.6.

Detail sidewalk paving brackets with the same width and elevation as the roadway paving bracket. Sidewalks are to be supported on abutment diaphragm or abutment backwalls and detailed to “float” along adjacent wingwalls.

For semi-integral and parapet abutments, avoid projections on the back of abutments that are less than 4’-6” below grade. If shallower projections are necessary, slope the bottom to minimize frost upheaval effects.

### 11.1.1 Integral Abutments

An integral abutment consists of an abutment stem supported by a single line of piles. The superstructure girders or slab bear on the stem. An abutment diaphragm is poured with the deck and encases the girders. The diaphragm is connected to the stem, making the superstructure integral with the abutment. Figure 11.1.1.3 shows typical integral abutment cross-section details and reinforcement. Figure 11.1.1.4 shows typical partial elevation details and reinforcement. The reinforcement in these figures is typical for an integral abutment design based on the Integral Abutment Reinforcement Design Guide found in this section. For abutments that do not meet the design guide criteria, these figures may not accurately reflect the final abutment design.

### Geometry

Use a minimum thickness of 3 feet for the abutment stem. For skewed bridges, increase the abutment thickness to maintain a minimum of 5” between the beam end and the approach slab seat (See Figure 11.1.1.3). Set the height of the abutment stem to 5 feet on the low side of the bridge, with 3 feet below grade, and 2 feet exposed (See Figure 11.1.2). (Note that the 4'-6" minimum depth below grade requirement for abutment footings does not apply to integral abutment stems.)

Orient H-piling such that weak axis bending occurs under longitudinal bridge movements. Limit the use of CIP piling to bridges 150 feet or less in length. Minimum pile penetration into abutment stem is 2'-6”. Avoid using 16” CIP and HP 14x73 piles because of limited flexibility.

Provide a 2'-0” x 2'-0” corner fillet on the back face of the wingwall/abutment connection.

Limit the length of the wingwall cantilever to 14 feet measured from the back face of abutment to the end of the wingwall. Place a permissible horizontal construction joint in the wingwall at the elevation of the
abutment stem/diaphragm interface, running the entire length of the wall. Also place a permissible vertical construction joint where the wingwall connects to the abutment fillet, running from the bridge seat to the top of the wingwall. Show membrane waterproofing along the inside face of construction joints. This gives the contractor the option of casting the upper portion of the wingwall separately or with the diaphragm and deck (See Figure 11.1.1.1).

![Wingwall Plan View](image1)

**Permissible Wingwall Construction Joints Detail**  
*Figure 11.1.1.1*

Tie the approach panel to the bridge with dowel bars that extend at a 45 degree angle out of the diaphragm through the paving bracket seat and bend horizontally 6 inches below the top of the approach panel. (See bar S1905E, Figure 11.1.1.3) Include a ½ x 7 inch bituminous felt strip on the bottom of the paving bracket.

For all concrete barriers that extend beyond the end of the bridge deck, provide a joint at the abutment/approach panel interface. Tool or vee-groove the joint from the deck surface to 12 inches above the deck surface. Through-trowel or saw cut a 1/8 inch gap in the remainder of the barrier and tool or vee-groove the surface. Seal the joint with an approved sealant. End reinforcing steel a minimum of two inches from the joint (see Figure 11.1.1.2).
Barrier Joint Details at Approach Panel

Figure 11.1.1.2

Verify that the roadway designer has included approach panel details for a jointless abutment in the grading plan when an integral abutment is used.
Figure 11.1.1.3

NOTES

1. MEMBRANE WATERPROOFING SYSTEM PER Mn/DOT SPEC 2481.3B
2. DISTANCE TO APPROACH PANEL DOWEL BAR
3. SHORTEN LENGTH AS NEEDED FOR RECTANGULAR BEAMS
4. TIE BAR TO TOP MAT
5. H-PILE SHOWN SIMILAR FOR CIP
6. CONCRETE BEAM SHOWN SIMILAR FOR STEEL BEAM

TYPICAL INTEGRAL ABUTMENT CROSS SECTION
Figure 11.1.1.4

INTEGRAL ABUTMENT REINFORCEMENT

<table>
<thead>
<tr>
<th>BAR</th>
<th>SHAPE</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1901E</td>
<td>STEM STIRRUP</td>
<td></td>
</tr>
<tr>
<td>A1902E</td>
<td>STEM HORIZONTAL</td>
<td></td>
</tr>
<tr>
<td>A1903E</td>
<td>PILE TIE</td>
<td></td>
</tr>
<tr>
<td>A1904E</td>
<td>STEM B.F. VERTICAL</td>
<td></td>
</tr>
<tr>
<td>A1905E</td>
<td>STEM SEAT TIE</td>
<td></td>
</tr>
<tr>
<td>A1906E</td>
<td>STEM F.F. VERTICAL</td>
<td></td>
</tr>
<tr>
<td>S1901E</td>
<td>DIAPHRAGM TRANSVERSE</td>
<td></td>
</tr>
<tr>
<td>S1902E</td>
<td>DIAPHRAGM F.F., HORIZ.</td>
<td></td>
</tr>
<tr>
<td>S1903E</td>
<td>DIAPHRAGM B.F., HORIZ.</td>
<td></td>
</tr>
<tr>
<td>S1904E</td>
<td>DIAPL./DECK TIE</td>
<td></td>
</tr>
<tr>
<td>S1905E</td>
<td>DIAPL./APPROACH TIE</td>
<td></td>
</tr>
<tr>
<td>S1906E</td>
<td>DIAPL./FILLET TIE</td>
<td></td>
</tr>
<tr>
<td>S1907E</td>
<td>DIAPHRAGM F.F., HORIZ.</td>
<td></td>
</tr>
<tr>
<td>S3508E</td>
<td>DIAPHRAGM THRULL BEAMS</td>
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</tr>
</tbody>
</table>

NOTES:
- ELEVATION DOES NOT SHOW HORIZONTAL BARS THROUGH BEAMS
- ELEVATION DOES NOT SHOW DECK AND APPROACH PANEL REINFORCEMENT

TYPICAL INTEGRAL ABUTMENT PARTIAL ELEVATION
**Integral Abutment Reinforcement Design Guide**

Integral abutment reinforcement may be designed using the following guidance on beam and slab span bridges where all of the following criteria are met:

- All requirements of Articles 11.1 and 11.1.1 of this manual are met
- Beam height \( \leq 72" \)
- Beam spacing \( \leq 13'-0" \)
- Pile spacing \( \leq 11'-0" \)
- Pile capacity \( \phi R_n \leq 165 \) tons
- Maximum abutment stem height \( \leq 7'-0" \)
- Deck thickness plus stool height \( \leq 15.5" \)

For beam heights that fall in between current Mn/DOT prestressed beam sizes (i.e. steel beams), use the values corresponding to the next largest beam height in the tables. Detail reinforcement using Figures 11.1.1.3 and 11.1.1.4.

For abutment stem shear reinforcement use #19 bars spaced at a maximum of 12” between piles along the length of the abutment. These bars are designated A1901E and A1905E in Figures 11.1.1.3 and 11.1.1.4.

For abutment stem back face vertical dowels, select bar size, spacing and length from Table 11.1.1.1. Embed dowels 4 ft into the stem. These bars are designated A_04E in Figures 11.1.1.3 and 11.1.1.4. Space A_04E dowels with the abutment stem shear reinforcement (A1901E) between piles.

---

**Table 11.1.1.1 Abutment Stem Vertical Dowels (A_04E) Minimum Required Bar Size and Length**

<table>
<thead>
<tr>
<th>Beam Size (in)</th>
<th>Bar Size &amp; Max Spacing</th>
<th>Bar Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>#16 @ 12”</td>
<td>*</td>
</tr>
<tr>
<td>18</td>
<td>#19 @ 12”</td>
<td>*</td>
</tr>
<tr>
<td>22</td>
<td>#19 @ 12”</td>
<td>*</td>
</tr>
<tr>
<td>27</td>
<td>#19 @ 12”</td>
<td>5'-6&quot;</td>
</tr>
<tr>
<td>36</td>
<td>#22 @ 12”</td>
<td>6'-3&quot;</td>
</tr>
<tr>
<td>45</td>
<td>#22 @ 12”</td>
<td>7'-0&quot;</td>
</tr>
<tr>
<td>54</td>
<td>#19 @ 6”</td>
<td>7'-6&quot;</td>
</tr>
<tr>
<td>63</td>
<td>#19 @ 6”</td>
<td>7'-6&quot;</td>
</tr>
<tr>
<td>72</td>
<td>#19 @ 6”</td>
<td>7'-6&quot;</td>
</tr>
</tbody>
</table>

* Hook bar around uppermost B.F. horizontal bar in diaphragm
For abutment stem front face vertical dowels, use #16 bars spaced at a maximum of 12” between beams. These bars are designated A1606E in Figures 11.1.1.3 and 11.1.1.4. Space A1606E with abutment diaphragm transverse reinforcement (S1601E).

For abutment stem front and back face horizontal reinforcement, use #19 bars spaced at a maximum of 12”. These bars are designated A1902E in Figures 11.1.1.3 and 11.1.1.4. Account for changes in abutment seat height by varying bar spacing or the number of bars.

For the abutment stem top and bottom longitudinal bars, use 4-#19 bars on the top and bottom faces of the stem. These bars are designated A1902E in Figures 11.1.1.3 and 11.1.1.4.

Include 2-#13 pile ties on each side of each pile. These bars are designated A1303E in Figures 11.1.1.3 and 11.1.1.4.

For abutment diaphragm transverse reinforcement, use #16 bars spaced at a maximum of 12” between beams. These bars are designated S1601E in Figures 11.1.1.3 and 11.1.1.4. Space S1601E with abutment stem front face vertical dowels (A1606E).

For abutment diaphragm deck ties, approach panel ties and fillet ties, use #19 bars spaced at a maximum of 12” between beams. These bars are designated S1904E, S1905E and S1906E, respectively in Figures 11.1.1.3 and 11.1.1.4. Space with abutment stem front face vertical dowels. Place two additional S1904E diaphragm deck ties at equal spaces at the end of each beam.

Provide 1-#13 horizontal bar in the fillet area of the abutment diaphragm that runs the width of the abutment. This bar is designated S1307E in Figures 11.1.1.3 and 11.1.1.4.

For abutment diaphragm front face and back face horizontal reinforcement, use equally spaced #19 bars. These bars are designated S1902E and S1903E, respectively in Figures 11.1.1.3 and 11.1.1.4. Determine the number of bars using Table 11.1.1.2.

For abutment diaphragms of concrete slab bridges, provide a minimum of two #19 bars in both the front face (S1902E) and back face (S1903E) with a maximum spacing of 12 inches.
Table 11.1.1.2 Abutment Diaphragm Horizontal Bars (S1902E & S1903E)
Minimum Required Number of #19 Bars, Each Face

<table>
<thead>
<tr>
<th>Beam Size (in)</th>
<th>Beam Spacing (ft)</th>
<th>≤ 9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
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<td>7</td>
<td>8</td>
<td>9</td>
<td></td>
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Integral Abutment General Design/Analysis Method

Design piling for axial loads only. Assume that one half of the approach panel load is carried by the abutment. Distribute live load over the entire length of abutment. Apply the number of lanes that will fit on the superstructure adjusted by the multiple presence factor. Use a minimum of four piles in an integral abutment.

For integral abutments that do not meet the Integral Abutment Reinforcement Design Guide criteria found in this section, use the methods outlined in this section to design the reinforcement.

Design vertical shear reinforcement in the abutment stem for the maximum factored shear due to the simple span girder reactions. Consider the stem to act as a continuous beam with piles as supports.

Design abutment stem backface vertical dowels for the passive soil pressure that develops when the bridge expands. Assume the abutment stem acts as a cantilever fixed at the bottom of the diaphragm and free at the bottom of the stem. Referring to Figure 11.1.1.5, determine the passive pressure \( p_p \) at the elevation of the bottom of the diaphragm and apply as a uniform pressure on the stem.

\[
p_p = k_p \cdot \gamma \cdot h_{soil}
\]

\[
k_p = \tan^2 \left(45 + \frac{\phi}{2}\right)
\]

\( \phi \) = angle of internal friction of the backfill material (use 30 degrees)
Then design for a moment $M_{up}$ equal to:

$$M_{up} = \gamma_{EH} \cdot \left( \frac{p_p \cdot h_{stem}^2}{2} \right)$$

A load factor for passive earth pressure is not specified in the LRFD specifications. Use the maximum load factor for active earth pressure, $\gamma_{EH} = 1.50$.

Design abutment stem front and back face horizontal bars for the passive soil pressure which results when the bridge expands. Consider the stem to be a continuous beam with piles as supports and design for a moment of:

$$M_{up} = \gamma_{EH} \cdot \left( \frac{w_p L^2}{10} \right)$$

$w_p$ = passive pressure calculated at the elevation of the bottom of abutment diaphragm and applied as a uniform pressure on the abutment stem

$p_p \cdot h_{stem}$

$L$ = pile spacing
Design abutment stem top and bottom horizontal bars for vertical loads due to girder reactions. Consider the stem to be a continuous beam with piles as supports.

Similar to abutment stem, design abutment diaphragm horizontal bars for the passive soil pressure which results when the bridge expands. For this case, consider the diaphragm to be a continuous beam with the superstructure girders as supports.

For size and spacing of all other abutment diaphragm bars, refer to the *Integral Abutment Reinforcement Design Guide*.

**11.1.2 Semi-Integral Abutments**

Semi-integral abutments are similar to integral abutments in that the superstructure and approach panel are connected and move together. Unlike integral abutments, the superstructure is supported on bearings that allow movement independent from the abutment stem. The abutment stem is stationary and is supported by a spread footing or a pile cap on multiple rows of piles. Figure 11.1.2.1 illustrates typical semi-integral abutment cross-section details and reinforcement.

**Geometry**

For skews greater than 30 degrees, provide a shear lug to reduce unwanted lateral movement during bridge expansion.

Refer to Figure 11.1.2 for minimum cover and clearance requirements.

Provide a minimum abutment stem thickness of 4'-0”.

Provide a 3” minimum horizontal gap between the abutment diaphragm lug and abutment stem.

Detail abutment/wingwall construction joints through the thickness of the abutment in a plane coincident with the back face of the wingwall.

Detail semi-integral abutments with a drainage system behind the wall (Detail B910). Outlet the 4 inch drains through wingwalls and backslopes.

Detail semi-integral abutment seat and pedestals in accordance with Article 11.1.3 of this manual under **Geometry**.
Figure 11.1.2.1

TYPICAL SEMI-INTEGRAL ABUTMENT

1. MEMBRANE WATERPROOFING SYSTEM PER MV/DOT 248L.38 EXCEPT THE STRIP SHALL BE 24" WIDE TO ALLOW MOVEMENT.

2. PILE FOUNDATION SHOWN SIMILAR FOR SPREAD FOOTING

DETAIL "A"

REINFORCEMENT
Design/Analysis
For single span bridges, provide fixity at one of the abutments.

Design semi-integral abutment stem, footing, and piles in accordance with Article 11.1.3 of this manual under Design/Analysis, except modify the Construction Case 1 loading as follows:

Construction Case 1a – Strength I (0.90DC+1.00EV+1.5EH+1.75LS)
Abutment stem has been constructed and backfilled, but the superstructure and approach panel are not in place. Use minimum load factors for vertical loads and maximum load factors for horizontal loads. Assume a single lane (12 ft width) of live load surcharge (LS) is acting on abutments less than 100 ft long measured along the skew. Apply two lanes of LS for abutments longer than 100 ft.

Construction Case 1b – Strength I (0.90DC+1.00EV+1.5EH+1.75LS)
Abutment has been constructed and the superstructure is in place. All of the backfill has been placed, but the approach panel has not been constructed. Use minimum load factors for vertical loads and maximum load factors for horizontal loads. Assume a single lane (12 ft width) of live load surcharge is acting on abutments less than 100 ft long measured along the skew. Apply two lanes of LS for abutments longer than 100 ft.

Design abutment diaphragm front and back face horizontal bars for the passive soil pressure which results when the bridge expands.

Semi-integral abutment diaphragm vertical bars are designed to resist the passive pressure that develops when the bridge expands. Assuming the diaphragm lug acts as a cantilever fixed at the bottom of the diaphragm, use #16E bars in the diaphragm lug to resist the moment that develops.

Semi-integral abutment diaphragm horizontal reinforcement can be designed using the Integral Abutment Reinforcement Design Guide found in this section, provided all of the criteria for the design guide are met. When using this guide for semi-integral abutments, the stem height requirement may be ignored. Design front and back face horizontal bars using Table 11.1.1.2, and place 4 additional #19 bars in the diaphragm lug. (See Figure 11.1.2.1)
Parapet abutments have backwall or parapet elements that are separate from the end diaphragms in the superstructure. Low parapet abutments have total heights (including footing) of less than 15 feet. High parapet abutments have total heights greater than 15 feet. If the total height of the abutment is more than 40 feet, counterforts should be considered.

**Geometry**

For parapet abutments, include pedestals under bearings and slope the bridge seat between pedestals to provide drainage away from the parapet wall and bearings. A standard seat slope provides one inch of fall from the back of the seat to the front of the seat. In no case should the slope be less than 2 percent.

Limit the maximum pedestal height to about 9 inches. The minimum pedestal height is 2 inches (at the front of the pedestal). Set back pedestals 2 inches from the front face of the abutment.

Bridges with mask walls can develop a horizontal crack at the top of the bridge seat that extends horizontally into the wingwall. To prevent such cracks from occurring, detail the abutment/wingwall construction joint through the thickness of the abutment in a plane coincident with the back face of the wingwall.

For bridges without mask walls, place a 2’-0” corner fillet at the back face wingwall/abutment connection. Detail a construction joint for the wingwall/abutment connection at the end of the corner fillet and running vertically through the wingwall thickness. Show membrane waterproofing along the inside face of the joint.
Design/Analysis
For design of piling or footing bearing pressures, as a minimum consider the following load cases:

Construction Case 1 – Strength I (0.90DC+1.00EV+1.5EH+1.75LS)
Abutment has been constructed and backfilled, but the superstructure and approach panel are not in place. Use minimum load factors for vertical loads and maximum load factors for horizontal loads. Assume a single lane (12 ft width) of live load surcharge is acting on abutments less than 100 ft long measured along the skew. Apply two lanes of LS for abutments longer than 100 ft.

Construction Case 2 – Strength I (1.25DC)
Abutment has been constructed, but not backfilled. The superstructure has been erected, but approach panel is not in place. Use maximum load factor for dead load.

Final Case 1 – Strength I (1.25DC+1.35EV+0.90EH+1.75LL)
Bridge is complete and approach panel is in place. Use maximum load factors for vertical loads and minimum load factor applied to the horizontal earth pressure (EH).

Final Case 2 – Strength I (1.25DC+1.35EV+1.50EH+1.75LL)
Bridge is complete and approach panel is in place. Use maximum load factor for all loads.

Design abutments for active pressure using an equivalent fluid weight of 0.033 kcf. A higher pressure may be required based on soil conditions. Neglect passive earth pressure in front of abutments.

Use LRFD Table 3.11.6.4-1 for determination of live load surcharge equivalent soil heights. Apply live load surcharge only when there is no approach panel.

Assume that one half of the approach panel load is carried by the abutment.

Distribute superstructure loads (dead load and live load) over the entire length of abutment. For live load, apply the number of lanes that will fit on the superstructure adjusted by the multiple presence factor.

For resistance to lateral loads, see Article 10.2 of this manual to determine pile resistance in addition to load taken by battering.
Design footing thickness such that no shear reinforcement is required. Performance of the Service 1 crack control check per LRFD 5.7.3.4 is not required for abutment footings.

Design abutment stem and backwall for horizontal earth pressure and live load surcharge loads. For stem and backwall crack control check, assume a Class 1 exposure condition ($\gamma_e = 1.00$).

11.1.3.1 Low Abutments

Low abutments shall have vertical contraction joints at about a 32 foot spacing. (See Detail B801.) A drainage system behind the stem need not be provided for low abutments. Figure 11.1.3.1.1 contains typical dimensions and reinforcing for low parapet abutments.

11.1.3.2 High Abutments

High abutments shall have vertical construction joints (with keyways) at about a 32 foot spacing.

Detail high abutments with a drainage system (Detail B910). Outlet the 4 inch drains through wingwalls and backslopes. Granular backfills at railroad bridge abutments typically includes perforated pipe drains. Figure 11.1.3.2.1 illustrates typical high abutment dimensions and reinforcing.
Figure 11.1.3.1.1

CONCRETE WEARING COURSE (3U17A) AND BRIDGE SLAB CONCRETE (3Y36) INCLUDED IN SUPERSTRUCTURE QUANTITIES.

PERMISSIBLE CONST. JT. & 2" X 6" KEYWAY

MEMBRANE WATERPROOFING SYSTEM PER Mn/DOT 2481.3B

FILE FOUNDATION SHOWN SIMILAR FOR SPREAD FOOTING

TYPICAL LOW PARAPET ABUTMENT

DETAILS

REINFORCEMENT
CONCRETE WEARING COURSE (3U17A) AND BRIDGE SLAB CONCRETE (3Y36) included in superstructure quantities.

Figure 11.1.3.2.1

DETAILS

1. MEMBRANE WATERPROOFING SYSTEM PER Mn/DOT 240.3B

2. PILE FOUNDATION SHOWN SIMILAR FOR SPREAD FOOTING

TYPICAL HIGH PARAPET ABUTMENT

CONST. JT. & 2" X 4" KEYWAY

PERMISSIBLE CONST. JT. & 2" X 6" KEYWAY

STRUCTURAL CONCRETE (3Y43)

STIRRUP CONCRETE (H43)

PILE CUTOFF

1'-4" MIN.

3" CLR. IN FIG. (TYP.)

BARS TO BE DESIGNED

3" CLR. (TYP.)

"16E TIE - PULL UP TO 2" CLR.

"19 TOP & BOT. LONG. BARS 1'-0" MAX. SPG.

"19E

"16E TIE

"16E

"19E

"16E

"16E

"16E

"16E

"16E

"16E

"16E

"16E
11.1.4 Wingwalls

The intended layout for the wingwalls will be provided in the preliminary plans.

Where wingwalls are oriented parallel to the centerline of the roadway, sidewalk and curb transitions should generally not be located adjacent to wingwalls.

The maximum cantilever beyond the edge of footing for wingwalls is 12 feet. For cantilevers up to 8 feet, no design is required if the following guidelines are used:

- wingwall thickness = 1’-6”
- maximum rustication depth = 2”
- horizontal bars are located inside or outside of the vertical bars
- maximum height of cantilever = 13’-6”
- horizontal back face reinforcement consists of #16 bars at 12 inches maximum spacing

For cantilevers greater than 8 feet in length, a design must be completed.

Avoid steep vertical offsets between wingwall footings by separating footings with a 1:1 soil slope or by using a single footing to support the entire wingwall.

With wingwalls over 20 feet long, separate footings with different elevations may be required. Assume soil pressures between abutment and wingwall footing are equally distributed to both footings. With spread footings, limit the bottom of footing elevation difference to 5 feet. With pile foundations, the distance between footing segments should be greater than 1.5 times the elevation difference between footings. Limit the cantilever (beyond the end of the footing) of wingwalls to 6 feet with separate wingwall footings.

Vertical construction joints shall be provided on wingwalls over 32 feet long.

Avoid horizontal wingwall construction joints unless hidden by other horizontal details. Joints between the barrier and wingwall tend to become visible over time due to water being carried through the construction joint by capillary action.

Provide reinforcement through the construction joint at the intersection of the wing and abutment wall to transfer wingwall loads to the abutment.
Within the plan set provide wingwall pile loads if they are less than 80% of the loads in the main portion of the abutment. When listing the total length of piling for an abutment and a separate wingwall, check if the wingwall pile needs to be longer than the abutment piles.

Figures 11.1.4.1 through 11.1.4.5 contain details and tables that can be used to determine the length of straight and 45º wingwalls. Guidance is provided for parapet and pile bent abutments.
### WINGWALL LENGTHS

<table>
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<tr>
<th>A-B VERT.HT.</th>
<th>SQUARE</th>
<th>10° RDWY SKEW</th>
<th>20° RDWY SKEW</th>
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<tr>
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<td>5'-0&quot; MIN.</td>
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### NOTES:
1. A DISTANCE OF 2'-0" FROM EDGE OF DECK TO EDGE OF SLOPE PAVING WAS USED IN CALCULATING THE WINGWALL LENGTHS.
2. WINGWALL LENGTHS OVER 8'-0" SHOULD HAVE FOOTINGS.
### Table

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<th>A-B VERT. HT.</th>
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**Figure 11.1.4.2**

**Parallel Wingwalls**

*With Parapet Abutments*

*Figure 11.1.4.2*
Figure 11.1.4.3

**WINGWALL LENGTHS**

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</table>

**NOTES:**

1. A DISTANCE OF 2'-0" FROM EDGE OF DECK TO EDGE OF SLOPE PAVING WAS USED IN CALCULATING THE WINGWALL LENGTHS.
2. WINGWALL LENGTHS OVER 8'-0" SHOULD HAVE FOOTINGS.
## Straight Wingwalls

*(With Pile Bent Abutments)*

**Figure 11.1.4.4**

### Table: Wingwall Lengths

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</table>

**Notes:**
1. A distance of 2'-0" from edge of deck to edge of wingwall lengths.
2. Wingwall lengths over 0'-0" should have footings.
Figure 11.4.5

WINGWALL LENGTHS

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<th>SQUARE</th>
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<td>16'-5&quot;</td>
<td>20'-11&quot;</td>
<td>26'-3&quot;</td>
</tr>
</tbody>
</table>

NOTES:

1. A distance of 2'-0" from edge of deck to edge of slope paving was used in calculating the wingwall lengths.
2. Wingwall lengths over 8'-0" should have footings.
3. (Area inside dashed line only) Distance along front face of wingwall governed.
Figure 11.1.4.6

Parallel Wingwalls

(With Integral Abutments)
In most cases approach panels are a roadway pay item. Inform the roadway designers of the appropriate approach panel detail to include in the roadway plans (for a jointless bridge or for a bridge with expansion joints). Also coordinate curb and median transitions with roadway designers.

Provide 8 inches of width for the abutment blockout or paving bracket, which supports the approach panel. Place the paving bracket not less than 1’-3” below the top of roadway surface. The reinforcement in the end block or paving block is shown in Figure 11.1.5.1.

Bridge Approach Treatment
For Mn/DOT repair projects and other projects where no separate grading plans are prepared, make sure that bridge approach treatments are consistent with roadway Standard Plan 5-297.233.

Bridge Approach Panel
Details for bridge approach panels for concrete and bituminous roadways are provided on the roadway Standard Plans 5-297.223 – 5-297.232. Use a concrete wearing course on approach panels when the bridge deck has a concrete wearing course. The wearing course will be placed on the bridge superstructure and the approach panels at the same time. Include the wearing course quantity for the approach panels in the superstructure summary of quantities.
11.2 Piers

A wide variety of pier types are used in bridge construction. The simplest may be pile bent piers where a reinforced concrete cap is placed on piling. A more typical pier type is a cap and column pier. Columns supported on individual footings support a common cap. The spacing of columns depends on the superstructure type, the superstructure beam spacing, and the size of the columns. A typical cap and column pier for a roadway may have from three to five columns. At times wall piers may be used to support superstructures. Where extremely tall piers are required, hollow piers may be considered. Specialty bridges such as segmental concrete bridges may use double-legged piers to reduce tie down reactions during segmental construction.

11.2.1 Geometrics

To facilitate the use of standard forms, detail round and rectangular pier columns and pier caps with outside dimensions that are multiples of 2 inches. As a guide, consider using 2’-6” columns for beams 3’-0” or less in depth, 2’-8” columns for beams 3’-1” to 4’-0”, 2’-10” columns for beams 4’-1” to 5’-0”, and 3’-0” columns for beams over 5’-0” unless larger columns are necessary (for strength or for adequate bearing area).

When laying out piers, consider the economy to be gained from reusing forms (both standard and non-standard) on different piers constructed as part of a single contract.

Dimension piles, footing dimensions, and center of columns to working points.

For pier caps (with cantilevers) supported on multiple columns, space the columns to balance the dead load moments in the cap.

Label the ends of piers (South end, North end, etc.).

11.2.2 Columns

The minimum column diameter or side of rectangular column is 2’-6”.

11.2.3 Cap

Slope pier caps in a straight line and utilize concrete pedestal beam seats when possible. Pedestals shall be set back at least 1 1/2 inches from the edge of cap and be no taller than 9 inches. Consider omitting pedestals if their height is less than 1 inch.

Choose a pier cap width and length that is sufficient to support bearings and provide adequate edge distances. As a guide, choose a pier cap depth equal to 1.4 to 1.5 times the width.
The bottom of the pier cap should be approximately parallel to the top. Taper cantilever ends about $\frac{1}{3}$ of the depth of the cap. When round pier columns are required, use rounded pier cap ends as well. The ends of pier caps for other types of pier columns should be flat. Detail solid shaft (wall) piers with rounded ends for both the cap and shaft.

**Integral Steel Box Beam Pier Caps**
Avoid the use of steel box beam pier caps whenever possible. Conventional concrete pier caps or open plate girder pier caps are preferred.

To ensure that components are constructible, review the design details of box beam pier caps with the Fabrication Methods Unit and the Structural Metals Inspection Unit early in the plan development process.

The minimum dimensions of a box pier cap are 3’-0” wide by 4’-6” high. Make access openings within the box as large as possible and located to facilitate use by inspection personnel. The minimum size of access openings in a box pier cap is 18” x 27” (with radius corners.).

Provide access doors near each end. If possible, locate the door for ladder access off of the roadway. Orient the hinge for the access doors such that doors swing away from traffic. Access doors can be placed on the side of box pier caps if they are protected from superstructure runoff. If not, locate in the bottom of the cap. Bolt the frame for the door to the cap in accordance with Bridge Detail Part I B942.

Bolted internal connections are preferred to welded connections. Fillet welds are preferred to full penetration welds.

Avoid details that may be difficult to fabricate due to clearance problems. Assume that welders need an access angle of at least 45 degrees and require 18 inches of clear working distance to weld a joint. The AISC Manual of Steel Construction contains tables with entering and tightening clearance dimensions for bolted connections.

Paint the interior of boxes for inspection visibility and for corrosion protection. Provide drainage holes with rodent screens at the low points of the box.

**Piers Adjacent to Railways**
Piers located within 50 feet of the centerline of railroad tracks are required to have crash walls incorporated into their design unless they are protected as specified in LRFD 3.6.5.1.
Piers located within 25 feet of the centerline of railroad tracks must either be of “heavy construction” or have crash walls.

A pier is considered to be of “heavy construction” when it meets all of the following:

- The cross-sectional area of each column is a minimum of 30 square feet.
- Each column has a minimum dimension of 2.5 feet.
- The larger dimension of all columns is parallel to the railroad track.

Crash walls must meet the following geometric requirements:

- Top of crash wall shall extend a minimum of:
  - 6 feet above top of railroad track when pier is between 12 feet and 25 feet from centerline of tracks.
  - 12 feet above top of railroad track when pier is 12 feet or less from centerline of tracks.
- Bottom of crash wall shall extend a minimum of 4 feet below ground line.
- Crash wall shall extend one foot beyond outermost columns and be supported on footing.
- Face of crash wall shall be located a minimum of 6 inches outside the face of pier column or wall on railroad side of pier.
- Minimum width of crash wall is 2.5 feet.
- Minimum length of crash wall is 12 feet.

Piers of “heavy construction” and crash walls adjacent to railroad tracks shall be designed for a minimum load of 400 kips.

**Piers Adjacent to Roadways**

Piers located within 30 feet of the edge of roadway are required to have crash walls incorporated into their design unless they are protected as specified in LRFD 3.6.5.1.

Crash walls adjacent to roadways shall be designed for a minimum load of 400 kips.

**11.2.5 Design and Reinforcement**

Include a standard hook at each end of all footing longitudinal and transverse reinforcement.

Use 90 degree standard hooks to anchor the dowel bars in the footing/column connection. Show the lap splice length for bent dowels and check development length of hooked end of dowel bar at
footing/column interface. Size dowel bars one size larger than column vertical reinforcement when the dowel bar is detailed to the inside of the column vertical.

Provide the dimensions between the center of column dowel patterns and the nearest working points.

To simplify construction, detail vertical column reinforcement to rest on top of the footing. Columns with a diameter less than or equal to 42 inches shall use spiral reinforcement. The spiral shall have a 3-inch pitch and be No. 13E. Extend spirals no less than 2 inches into the pier cap. Use Table 5.2.2.3 to compute the weight of column spiral reinforcement.

Round columns over 42 inches in diameter and square or rectangular columns shall be designed with tied reinforcement. Ties no smaller than No. 10E shall be used when the column vertical bars are No. 32E or smaller. No. 13E or larger ties shall be used for No. 36E, No. 43E, No. 57E, and bundled column vertical bars. The maximum spacing for ties is 12 inches. Place the first tie 6 inches from the face of the footing, crash wall, or pier cap.

Use standard hooks to develop the top longitudinal reinforcement at the ends of pier caps.

Provide 2 inches minimum clear distance between anchor rods and longitudinal reinforcement bars. For piers without anchor rods, provide a single 6-inch minimum opening between longitudinal reinforcement bars to facilitate concrete placement.

The maximum size of pier cap stirrups is No. 16E. Use open stirrups unless torsion loads are large enough to require closed stirrups. If necessary, use double stirrups to avoid stirrup spacing of less than 4 inches.

Provide No. 13E or No. 16E ties in both directions under bearing assemblies (6-inch to 8-inch spacing). The clear distance from the top of reinforcement to the top of the pier cap shall be no less than 2 inches. Detail ties to clear bearing anchor rods by a minimum of 2 inches.

11.2.6 Miscellaneous
Show an optional construction joint at the top of columns. All construction joints should be labeled and the size of keyways identified.
Detail a \(\frac{3}{4}\) inch V-strip on the bottom of pier cap ends to prevent water from migrating on to substructure components.

Provide a vertical open joint in pier caps that have a total length exceeding 100 feet. The design may dictate that additional pier cap joints are necessary to relieve internal forces.

11.2.6.1 Pile Bent

Single line pile bent piers shall be constructed with piles no smaller than 16-inch diameter CIP piles. Refer to Section 10.6 and Figure 10.6.1 for discussion of the unsupported pile length for pile bent piers.

The preliminary plan may specify that a wall is to be provided which encases the piles from the bottom of the cap to the flowline. The wall provides stability and protects the piling from debris. In this case, H-piles or CIP piles less than 16 inches in diameter are sometimes specified.

11.3 Retaining Walls

Retaining wall designs need to consider several parameters. These parameters include:

- Height of the wall
- Geometry of the wall (curved or straight)
- Type of material retained
- Geometry of the backfill (level or sloped)
- Magnitude of live load surcharge
- Whether or not traffic barriers will be incorporated into the top of the wall (vehicle collision loads)
- Whether or not noise walls will be supported on the wall
- Location of the water table
- Quality of subgrade material (supported on spread footings or pile foundations).

11.3.1 Cantilever Retaining Walls

In many cases a conventional reinforced concrete retaining wall is the appropriate solution for a project. For wall heights up to 29 feet, use standard details. Mn/DOT standard cantilever retaining wall designs and details (Bridge Standard Plans Fig. 5-395.200 through 5-395.212) are available for download from the Bridge Office web site.

11.3.2 Counterfort Retaining Walls

Counterforted retaining walls are economical for wall heights over 40 feet. Counterforted walls are designed to carry loads in two directions. Earth pressures are carried laterally with horizontal reinforcing to
thickened portions of the wall. The thickened portion of the wall contains the counterfort, which is designed to contain vertical reinforcement that carries the overturning loads to the foundation.

### 11.3.3 Anchored Walls

#### General
Anchored walls are used when the height of the earth to be retained by the wall is considerable and when all other types of retaining walls prove to be uneconomical. In order to reduce the section of the stem, an anchoring system is provided at the back of the wall. Anchoring is typically accomplished by embedding a concrete block “dead man” in earth fill and connecting it to the stem of the wall with anchor rods. Alternatively, the anchors may be incorporated into soil nails or rock bolts. The feasibility of using anchored walls should be evaluated on a case-by-case basis after all other types of retaining walls have been ruled out as an option.

#### Design and Construction Requirements
The design shall meet the current safety and movement requirements of Section 11.9 of AASHTO LRFD Bridge Design Specifications.

Construction shall be in accordance with Section 7 of AASHTO LRFD Bridge Construction Specifications, AASHTO LRFD Bridge Design Specifications and the Mn/DOT Standard Specifications for Construction.

### 11.3.4 Mechanically Stabilized Earth Walls

#### General
Mechanically stabilized earth walls are reinforced soil retaining wall systems that consist of vertical or near vertical facing panels, metallic or polymeric tensile soil reinforcement, and granular backfill. The strength and stability of mechanically stabilized earth walls derives from the composite response due to the frictional interaction between the reinforcement and the granular fill. Mechanically stabilized earth systems can be classified according to the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing. The AASHTO LRFD Bridge Design Specifications list three major types of mechanically stabilized earth walls according to facing type. They are:

1. Precast concrete panel (MSE) walls. An MSE wall, in Mn/DOT terminology, refers to the precast concrete panel walls. Technical Memorandum No. 03-16-MRR-06 shall be used for design and construction of these walls. An approved list of MSE wall systems is available from the Bridge Office website.
MSE walls may be used in lieu of conventional gravity, cantilever, or counterfort retaining walls. MSE walls offer some advantages when settlement or uplift is anticipated. In some cases MSE walls offer cost advantages at sites with poor foundation conditions. This is primarily due to the costs associated with foundation improvements such as piles and pile caps that may be required to support conventional wall systems.

In general, MSE walls shall not be used where:

- Two walls meet at an angle less than 70°.
- There is scour or erosion potential that may undermine the reinforced fill zone or any supporting footing.
- Walls have high curvature (radius less than 15 meters (50 feet)).
- Soil is contaminated by corrosive material such as acid mine drainage, other industrial pollutants, or any other condition which increases corrosion rate such as the presence of stray electrical currents.
- Sites where extensive excavation is required or sites that lack granular soils and the cost of importing suitable fill material may render the system uneconomical.
- Walls are along shorelines.
- Retaining walls support roadways unless an impervious layer is placed below the roadway surface to drain any surface water away from the reinforcement.
- There is potential for placing buried utilities within the reinforced zone.

The design of precast panel MSE walls shall meet all the requirements of the MSE Wall Technical Memorandum.

2. Modular Block Wall (MBW). The facing for this wall is made of small, rectangular concrete units that have been specially designed and manufactured for retaining wall applications. MBW designs shall meet the design requirements of the related Mn/DOT Technical Memorandum and the Standard sheets (5-297.640, 641, 643, 644, 645).

3. Cast-in-place (CIP) concrete facing with Reinforced Soil Slopes (RSS). The facing for this wall is cast-in-place concrete. This type of wall is not approved for use in Mn/DOT projects.
Prefabricated modular walls are gravity walls made of interlocking soil-filled concrete or steel modules or bins, rock filled gabion baskets, precast concrete units or modular block units (without soil reinforcement).

Prefabricated modular walls shall not be used under the following conditions:
- On curves with radius of less than 800 ft, unless the curve could be substituted by a series of chords
- Steel modular systems shall not be used where the ground water or surface runoff is acid contaminated or where deicing spray is anticipated.
- Heights greater than 8 feet.

Design and Construction Requirements
The design shall meet the current safety and movement requirements of Section 11.11 of AASHTO LRFD Bridge Design Specifications.

The construction shall be in accordance with Section 7 of AASHTO LRFD Bridge Construction Specifications, AASHTO LRFD Bridge Design Specifications and the Mn/DOT Standard Specifications for Construction.

Cantilevered Sheet Pile Walls
Cantilever sheet piling is used in many ways on bridge projects. Most often it is used to contain fill on a temporary basis for phased construction activities, as when existing embankments need protection or new embankments need to be separated from existing facilities during construction. Sheet piling is also used in the construction of cofferdams.

Most often hot rolled steel sheet piling is used for cantilevered sheet pile walls. Hot rolled sections are available from domestic and foreign sources. Domestically, only PZ22 and PZ27 sections are being rolled. Securing new domestic material may have a significant lead time. For many temporary applications, new material is not required and the contractor may have a supply of used sections.

For applications that are insensitive to water filtration through the interlocks, cold formed sections may be used. For railway applications,
check with the railroad as cold formed sections are not allowed by some railroads.

If temporary sheeting is anticipated for a project the following guidelines should be followed:

When significant quantities of sheet piling are anticipated, or when an anchored wall design is required, the wall shall be designed and detailed in the bridge plans, including the required section modulus and tie back forces. Also provide an informational schedule of quantities. Payment for the sheet pile wall shall be made as a stand alone pay item on a lump sum basis.

For most other instances, the amount and design of sheet pile used will often depend on the contractor’s operations. When it is anticipated that sheet pile will likely be used, show the approximate location of the sheet pile wall in the plan along with a construction note stating: Payment for sheet piling shall be considered incidental to other work.

Payment for sheet piling used for typical foundation excavations is described in the standard special provisions developed for Structure Excavation and Foundation Preparation and need not be shown in the plans.

Design charts suitable for use in preliminary design tasks and approximating depths for contractor designed sheeting have been assembled in this section. The charts are based on a number of assumptions and contain several limitations. Refer to Figure 11.3.8.1, Figure 11.3.8.2, and Tables 11.3.8.1 through 11.3.8.4.

**11.3.8 Design Charts of Cantilevered Sheet Pile Soil Retention Walls for Temporary Applications**

**DESIGN CHART ASSUMPTIONS**

1. A level ground surface has been assumed in front of and behind the sheet piling.
2. A moist soil unit weight of 120 pounds per cubic foot has been used through the retained height.
3. The unit weights used below the excavation depth were determined using the S.P.T. N–value vs. unit weight correlation given on the attached sheet. In the Cohesive Material Case, the average cohesion value I was divided by 100 to obtain an equivalent S.P.T. N-value and the correlation between S.P.T. N–value and unit weight was used.
4. A uniform vertical surcharge of 240 pounds per square foot has been used to account for the traffic live load.
5. In addition to the traffic surcharge load (which results in a uniform lateral distributed pressure of 80 pounds per square foot), an equivalent fluid pressure of 40 pounds per cubic foot has been used throughout the retained height.

6. Coulomb’s earth pressure theory has been used in the development of the Granular Material Case, assuming a wall friction angle of $\frac{1}{3}$ phi. The friction angles used were determined using the S.P.T N-value vs. friction angle correlation given on the attached sheet.

7. A minimum active earth pressure coefficient of 0.25 has been used below the retained height.

8. The factor of safety for embedment was obtained by reducing the calculated passive pressure diagram by 33%, which resulted in an increase in embedment ranging from 26% to 35% for the granular case and 43% to 80% for the cohesive case. To avoid over-reliance on the strength of only a few samples, the minimum embedment used in the charts was limited to no less than 75% of the retained height.

9. The required section modulus has been computed using 33% reduction on passive soil pressures and the relationship

$$f_b = 0.66 \cdot f_y,$$

where the sheet piling yield stress ($f_y$) is 38,500 pounds per square inch.

HOW TO USE THE DESIGN CHARTS

Cohesive Material Case

The average Cohesion Value I, to be used in the design chart shall be taken as the lower of the following:

i) The average of the C values, within the required embedment depth indicated on the nearest structure boring log(s).

ii) The average of the C values in the upper 50% of the required embedment depth.

In calculating the above averages, an embedment depth (typically equal to the exposed height) must first be assumed. If the resulting embedment depth obtained from the chart is different than that first assumed, re-calculate the average Cohesion Value using an adjusted embedment depth. Continue this process until the assumed embedment depth used in calculating the average Cohesion Value equals the resulting embedment depth from the chart to within one foot.

To enter the section modulus chart, use the average Cohesion values in the upper $\frac{1}{3}$ of the required embedment.
**Granular Material Case**

The average Standard Penetration Blow Count (N), to be used in the design chart for embedment in the Granular Material Case, shall be taken as the lower of the following:

i) The average of the N-values, within the required embedment depth, indicated on the nearest structure boring log(s).

ii) The average of the N-values in the upper 50% of the required embedment depth.

In calculating the above averages, an embedment depth (typically equal to the exposed height) must first be assumed. If the resulting embedment depth obtained from the chart is different than the first assumed, re-calculate the average Standard Penetration Blow Count using an adjusted embedment depth. Continue this process until the assumed embedment used in calculating the average Standard Penetration Blow Count equals the resulting embedment depth from the chart to within one foot.

To enter the section modulus chart, use the average N-values in the upper $\frac{1}{3}$ of the required embedment calculated above.

**Combined Material Case**

If the soil profile within the calculated embedment depth is a combination of granular and cohesive, divide the Cohesion values of each sample by 100 to determine the embedment using the granular material design chart.

**LIMITATIONS OF DESIGN CHARTS**

1. Any sheets not reaching their required embedment due to existing closed abutment footing must be restrained by developing an attachment to the existing abutment walls subject to the approval of the Engineer. This condition may sometimes also require wale support. For a maximum retained height of 12 feet and footing heel width less than or equal to 4 feet, these charts may be used provided the sheet adjacent to the existing abutment wall is connected to the abutment wall throughout the retained height.

2. The Design Charts are not applicable if the calculated sheet piling tip elevation falls in material with a Cohesive Value larger than 4500 pounds per square foot, an N-value larger than 45 blows, or rock, since the sheet piling may not get the necessary embedment to work as a cantilever.

3. If upon calculating the average soil strength for cohesive soils in the upper $\frac{1}{3}$ of the embedment, the lower soil strength in that $\frac{1}{3}$ is
larger than 2 times the upper soil strength in the same \( \frac{1}{3} \) of embedment, the section modulus in the chart should be amplified according to the following schedule:

<table>
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<tr>
<th>Lower Soil/Upper Soil Strength Ratio</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
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</thead>
<tbody>
<tr>
<td>Section Modulus Amplification Factor</td>
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<td>1.1</td>
<td>1.2</td>
<td>1.3</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Soil Strength Ratios should be rounded to the nearest whole number.

4. The charts attempt to provide a simple procedure to yield a quick design with adequate safety factor. Considering the extreme variability in soil conditions and in boring data interpretation, it is conceivable that the use of the charts can yield erroneous results. The engineer must always use good judgment in soil data interpretation and the use of the charts. If the chart design procedure results in a blank location on the chart, a cantilever design may still be possible with more rigorous analysis.

5. If any of the site conditions do not meet or are not consistent with the “Design Chart Assumptions”, the design charts are no longer applicable and should not be used.
Figure 11.3.8.1 – ASSUMED RELATIONSHIP OF BLOW COUNT AND FRICTION ANGLE, MOIST & SATURATED UNIT WEIGHT

\[ \phi = 21 \left[ e^{(\ln(N) + 4)^2/100} \right] \]

**BLOW COUNT VS FRICTION ANGLE \( \phi \)**

**BLOW COUNT VS UNIT WEIGHT**

Saturated Unit Weight = 105 (N) \( ^{0.07} \)

Moist Unit Weight = 95 (N) \( ^{0.095} \)
Figure 11.3.8.2 - Typical Sheet Pile Configuration
### Table 11.3.8.1 – TEMPORARY SHEET PILING DESIGN CHART – GRANULAR MATERIAL CASE

**REQUIRED SECTION MODULUS (in.³/ft.)**

<table>
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<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
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**AVERAGE COHESION “C” (PSF)**
## Table 11.3.8.2 – TEMPORARY SHEET PILING DESIGN CHART – GRANULAR MATERIAL CASE

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Section 11 concludes with three design examples. The examples are a high parapet abutment supported on piling, a retaining wall supported on a spread footing, and a three column pier.

11.4 Design Examples

11.4.1 High Parapet Abutment Design Example

This example illustrates the design of a high parapet abutment. After determination of dead, earth, and live load components, five load combinations are assembled (two strength load cases during construction, two strength load cases for the completed abutment, and a service load case for the completed abutment.) After which, the capacity of an assumed pile group is evaluated. Subsequently, the flexural design of the footing, stem, and backwall is presented. The shear capacity of the footing is also checked. A typical cross-section for the abutment is provided in Figure 11.4.1.1.

![Figure 11.4.1.1](image-url)
The design parameters for the example are:

1) This example is a continuation of the plate girder and expansion bearing design examples. The superstructure consists of a 9\(\frac{1}{2}\)" deck on 5 steel girders with a beam spacing of 11’-4” and a skew of 20°. The height of the backwall is 6’-9”.

2) The abutment is supported on cast-in-place piling (12-inch diameter).

3) The abutment supports a 20-foot long, 1-foot thick approach panel that is partially supported by the top of the backwall and partially supported by subgrade material.

Material and design parameters used in this example are presented in Table 11.4.1.1.

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</table>

### A. Evaluate Pile Bearing Capacity

The Bridge Construction Unit’s Recommendations for the foundations are referenced at the start of final design. The recommendations identify the appropriate design capacity and resistance factor to be used.

Nominal Capacity, \(Q_n = 225\) tons/pile

Resistance Factor, \(\phi = 0.45\)

Bearing Resistance, \(Q_r = \phi \cdot Q_n = 0.45 \cdot 225 = 101\) tons/pile

\[= 202\] kips/pile

Figure 11.4.1.2 shows a plan view of the abutment and includes an assumed pile layout for the example. Pile Rows I and II contain eight piles, while Row III contains seven piles. Avoid pile layouts that permit individual piles to go into tension.
B. Permanent Loads (DC & EV)

Calculate the dead loads:

Backwall:
\[ P_{bw} = 0.150 \cdot 1.50 \cdot 6.75 \cdot 54.27 = 82.4 \text{ kips} \]

Stem:
\[ P_{st} = 0.150 \cdot 4.5 \cdot 15.25 \cdot 54.27 = 558.6 \text{ kips} \]

Footing (to simplify load calculations, weight of step is included in stem):
\[ P_f = 0.150 \cdot (4 \cdot 11.0 + 4.25 \cdot 4) \cdot 55.33 = 506.3 \text{ kips} \]

Approach Panel (Assume half carried by the abutment):
\[ P_{ap} = 0.150 \cdot 1 \cdot \frac{20}{2} \cdot \frac{48}{\cos 20^\circ} = 76.6 \text{ kips} \]

Expansion Joint Block:
\[ P_{ejb} = 0.150 \cdot 1.25 \cdot 1.33 \cdot \frac{51.33}{\cos 20^\circ} = 13.6 \text{ kips} \]

Superstructure Dead Load (DC\(_1\) + DC\(_2\) reactions from the Plate Girder Example):
\[ P_{super} = (95 + 22) \cdot (5 \text{ girders}) = 585.0 \text{ kips} \]

Wingwall DL (Assume 5 feet of the wing walls beyond the footing are carried by the abutment. Also assume the corner fillet weight is balanced by the taper in the wing wall.):
\[ P_{wing} = 0.150 \cdot 2 \cdot 1.50 \left( \frac{6.5}{\cos 20^\circ} + 5 \right) \cdot (15.25 + 6.75 + 1.25) \\
= 124.7 \text{ kips} \]

Railing DL:
\[ P_{rail} = 0.439 \cdot 2 \left( 5 + \frac{6.5 + 0.67 + 1.33}{\cos 20^\circ} \right) = 12.3 \text{ kips} \]

Summing the dead loads,
\[ P_{DC} = 82.4 + 558.6 + 506.3 + 76.6 + 13.6 + 585.0 + 124.7 + 12.3 \\
= 1959.5 \text{ kips} \]

Calculate vertical earth pressure (EV) of fill above the footing:

On the Heel:
\[ P_{EV} = 0.120 \cdot (15.25 + 6.75) \cdot 6.5 \cdot \frac{48.0}{\cos 20^\circ} = 876.5 \text{ kips} \]
On the Toe:

\[ P_{EV} = 0.120 \cdot \left( \frac{3.75 + 1.75}{2} \right) \cdot 4 \cdot 55.33 = 73.0 \text{ kips} \]

Figure 11.4.1.3 summarizes the permanent loads and includes moment arms measured from the toe of the footing.

**C. Earth Pressure (EH)**

The active earth pressure values used for the equivalent fluid method (described in LRFD Article 3.11.5.5) range from 0.030 kcf to 0.035 kcf. Assuming a level backfill, Mn/DOT practice is to use:

\[ \gamma_{eq} = 0.033 \text{ kcf} \]

The respective horizontal active earth pressures at the top and bottom of the abutment are:

\[ P_{top} = 0 \text{ ksf} \]
P_{bottom} = \gamma_{eq} \cdot h = 0.033 \cdot 26.25 = 0.866 \text{ksf}

P_{EH} = 0.5 \cdot 0.866 \cdot 26.25 \cdot \left(\frac{48}{\cos 20^\circ}\right) = 580.6 \text{ kips}

The force acts at a location of \(\frac{1}{3}\) times the height of the load:

\[ 0.33 \cdot 26.25 = 8.66 \text{ ft} \]

Passive earth pressure in front of the abutment is neglected in the design. Figure 11.4.1.3 illustrates the horizontal earth pressure loading.

**D. Live Load Surcharge (LS) [3.11.6]**

The live load surcharge is applied to the abutment during construction. It represents construction activity on the fill behind the abutment prior to construction of the approach panel.

\[ \Delta p = \gamma_{eq} \cdot h_{eq} \]

From Table 3.11.6.4-1, use a surcharge height of 2.0 feet.

\[ \Delta p = 0.033 \cdot 2.0 = 0.066 \text{ kips/ft}^2 \]

Horizontal Resultant of LS is:

\[ P_{LS} = 0.066 \cdot 26.25 \cdot \frac{48}{\cos 20^\circ} = 88.5 \text{ kips} \]

Figure 11.4.1.3 illustrates the horizontal pressure loading associated with the live load surcharge.

**E. Live Load (LL)**

From the Plate Girder Design Example (Table 6.9.11) the maximum live load reaction without dynamic load allowance at the abutment is:

\[ R_{LL} = 66 + 42 = 108 \text{ kips/lane} \]

Coincident with live load on the superstructure, lane loading is applied to the approach panel. Use the same distribution that was used for dead load (assume that one half of the total load is carried by the abutment and the other half is carried in direct bearing to the subgrade away from the abutment):

\[ 0.64 \cdot 20 \cdot \frac{1}{2} = 6.4 \text{ kips/lane} \]

**[Table 3.6.1.1.2-1]**

For maximum loading, four lanes of traffic are placed on the superstructure and approach panel. The multiple presence factor for more than 3 design lanes is 0.65. For simplicity, add the live load from
the approach panel to the live load from the superstructure and apply the total at the centerline of bearing:
\[ P_{LL} = (108 + 6.4) \cdot 4 \cdot 0.65 = 297.4 \text{ kips} \]

Table 11.4.1.2 Vertical Load Components and Moments about Toe of Footing

<table>
<thead>
<tr>
<th>Load</th>
<th>Label</th>
<th>( P ) (kips)</th>
<th>Distance To Toe (ft)</th>
<th>Moment About Toe (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backwall</td>
<td>( P_{bw} )</td>
<td>82.4</td>
<td>-7.75</td>
<td>-638.6</td>
</tr>
<tr>
<td>Stem</td>
<td>( P_{st} )</td>
<td>558.6</td>
<td>-6.25</td>
<td>-3491.3</td>
</tr>
<tr>
<td>Footing</td>
<td>( P_{f} )</td>
<td>506.3</td>
<td>-7.41</td>
<td>-3751.7</td>
</tr>
<tr>
<td>Approach Panel</td>
<td>( P_{ap} )</td>
<td>76.6</td>
<td>-8.17</td>
<td>-625.8</td>
</tr>
<tr>
<td>Expansion Joint Block</td>
<td>( P_{ejb} )</td>
<td>13.6</td>
<td>-7.17</td>
<td>-97.5</td>
</tr>
<tr>
<td>Superstructure DL</td>
<td>( P_{super} )</td>
<td>585.0</td>
<td>-5.50</td>
<td>-3217.5</td>
</tr>
<tr>
<td>Railing</td>
<td>( P_{rail} )</td>
<td>12.3</td>
<td>-13.25</td>
<td>-163.0</td>
</tr>
<tr>
<td>Wingwall</td>
<td>( P_{wing} )</td>
<td>124.7</td>
<td>-14.10</td>
<td>-1758.3</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>1959.5</td>
<td></td>
<td><strong>-13743.7</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load</th>
<th>Label</th>
<th>( P ) (kips)</th>
<th>Distance To Toe (ft)</th>
<th>Moment About Toe (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill on Heel</td>
<td>( P_{(HEEL)} )</td>
<td>876.5</td>
<td>-11.75</td>
<td>-10298.9</td>
</tr>
<tr>
<td>Fill on Toe</td>
<td>( P_{(TOE)} )</td>
<td>73.0</td>
<td>-2.24</td>
<td>-163.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>949.5</td>
<td></td>
<td><strong>-10462.4</strong></td>
</tr>
<tr>
<td><strong>LL</strong></td>
<td><strong>P_{LL}</strong></td>
<td>297.4</td>
<td>-5.50</td>
<td>-1635.7</td>
</tr>
</tbody>
</table>

Table 11.4.1.3 Horizontal Load Components and Moments about Bottom of Footing

<table>
<thead>
<tr>
<th>Load</th>
<th>Type</th>
<th>Description</th>
<th>Label</th>
<th>( H ) (kips)</th>
<th>Distance to Toe (ft)</th>
<th>Moment About Toe (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EH</strong></td>
<td>Horizontal Earth Load</td>
<td>( P_{EH} )</td>
<td>580.6</td>
<td>8.66</td>
<td>5028.0</td>
<td></td>
</tr>
<tr>
<td><strong>LS</strong></td>
<td>Live Load Surcharge</td>
<td>( P_{LS} )</td>
<td>88.5</td>
<td>13.13</td>
<td>1162.0</td>
<td></td>
</tr>
</tbody>
</table>

**F. Select Applicable Load Combinations and Factors For Pile Design** [3.4.1]

Assemble the appropriate load factor values to be used for each of the load combinations. Load combinations for the Strength I Limit State are used. The load cases considered for the design example are:

All load modifiers = 1.0.

Strength I: Construction Case 1

\[ 0.90 \cdot DC + 1.00 \cdot EV + 1.5 \cdot EH + 1.75 \cdot LS \]
For this construction case, DC does not contain any superstructure or approach panel dead loads. It also assumes that the abutment is backfilled prior to superstructure erection.

Strength I: Construction Case 2

\[ 1.25 \cdot DC \]

For this construction case, DC includes the superstructure but does not include the approach panel. It assumes the superstructure is erected prior to the abutment being backfilled.

Strength I: Final Case 1

\[ 1.25 \cdot DC + 1.35 \cdot EV + 0.90 \cdot EH + 1.75 \cdot LL \]

This load case represents the completed structure with the minimum load factor for the horizontal earth pressure load.

Strength I: Final Case 2

\[ 1.25 \cdot DC + 1.35 \cdot EV + 1.50 \cdot EH + 1.75 \cdot LL \]

This load case represents the completed structure with the maximum load factor for the horizontal earth pressure load.

Table 11.4.1.4 contains the load factors that are used for each load component for each load case.

<table>
<thead>
<tr>
<th>Table 11.4.1.4 – Load Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
</tr>
<tr>
<td>DC</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
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<tr>
<td></td>
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<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>EV</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>EH</td>
</tr>
<tr>
<td>LS</td>
</tr>
<tr>
<td>LL</td>
</tr>
</tbody>
</table>
Table 11.4.1.5 lists the net vertical, horizontal, and moment forces that are applied to the pile group for each of the four load combinations.

<table>
<thead>
<tr>
<th>Strength I: Construction Case 1</th>
<th>Strength I: Construction Case 2</th>
<th>Strength I: Final Case 1</th>
<th>Strength I: Final Case 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Load P (kips)</td>
<td>Horizontal Load H (kips)</td>
<td>Moment about Toe M_toe (kip-ft)</td>
<td></td>
</tr>
<tr>
<td>2118</td>
<td>1026</td>
<td>-9797</td>
<td></td>
</tr>
<tr>
<td>2354</td>
<td>0</td>
<td>-16,397</td>
<td></td>
</tr>
<tr>
<td>4252</td>
<td>523</td>
<td>-29,641</td>
<td></td>
</tr>
<tr>
<td>4252</td>
<td>871</td>
<td>-26,624</td>
<td></td>
</tr>
</tbody>
</table>

Check Vertical Capacity of Pile Group

Determine the properties of the pile group. These properties include the number of piles, the location of the centroid or neutral axis with respect to the toe, and the moment of inertia of each pile row.

<table>
<thead>
<tr>
<th>Pile Group Properties</th>
<th>Row Number</th>
<th>Sum</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>Piles Per Row (N)</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Distance to Toe (d_toe)</td>
<td>1.25</td>
<td>6.0</td>
</tr>
<tr>
<td>N·d_toe</td>
<td>10.00</td>
<td>48.00</td>
</tr>
<tr>
<td>Neutral Axis of Pile Group to Toe (X_{NA})</td>
<td>(\sum N·d_toe)/ \sum N</td>
<td>6.71</td>
</tr>
<tr>
<td>Distance from Neutral Axis to Pile Row (d)</td>
<td>5.46</td>
<td>0.71</td>
</tr>
<tr>
<td>I = N·d^2</td>
<td>238.5</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.

The force in each pile row is found using:

\[
P\text{load} = \frac{P}{N} + \frac{M_{NA}}{I}
\]

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group. For Strength I: Construction Case I, the eccentricity about the toe is

\[e_{toe} = \frac{M_{toe}}{P} = \frac{-9797}{2118} = -4.63 \text{ ft}\]

Then the eccentricity about the neutral axis of the pile group is

\[e_{NA} = x_{NA} + e_{toe} = 6.71 - 4.63 = 2.08 \text{ ft}\]
The moment about the neutral axis of the pile group becomes
\[ M_{NA} = P \cdot e_{NA} = 2118 \times 2.08 = 4405 \text{ kip} - \text{ft} \]

Then Pile Load_{Row I} = \frac{2118}{23} + \frac{4405(5.46)}{589.4} = 132.9 \text{ kips/pile}

Pile Load_{Row II} = \frac{2118}{23} + \frac{4405(0.71)}{589.4} = 97.4 \text{ kips/pile}

Pile Load_{Row III} = \frac{2118}{23} + \frac{4405(-7.04)}{589.4} = 39.5 \text{ kips/pile}

The same calculations were carried out for the other load cases.

A summary of \(M_{NA}\) and the pile loads are provided in Table 11.4.1.7.

**Table 11.4.1.7 – Pile Reactions**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Pile Loads (kips/pile)</th>
<th>Moment about N.A. of pile group (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Row I</td>
<td>Row II</td>
</tr>
<tr>
<td>Strength I: Construction Case 1</td>
<td>132.9</td>
<td>97.4</td>
</tr>
<tr>
<td>Strength I: Construction Case 2</td>
<td>96.7</td>
<td>101.6</td>
</tr>
<tr>
<td>Strength I: Final Case 1</td>
<td>174.6</td>
<td>183.5</td>
</tr>
<tr>
<td>Strength I: Final Case 2</td>
<td>202.6</td>
<td>187.2</td>
</tr>
</tbody>
</table>

The largest pile load is 202.6 kips/pile (101.3 tons/pile), which is only 0.3% greater than the bearing resistance of 202 kips. Therefore, the pile layout is considered satisfactory for bearing.

**Check Lateral Capacity of Pile Group**

The maximum factored horizontal load from Table 11.4.1.5 is

\[ H = 1026 \text{ kips} \]

From Section 10.2, assume a lateral resistance of 18 kips/pile plus the resistance due to the two rows of battered piles.

\[ R_H = 23 \times 18 + 16 \times 202 \times \frac{3}{\sqrt{3^2 + 12^2}} = 1198 \text{ kips} > 1026 \text{ kips} \quad \text{OK} \]

**Pile Load Table for Plan**

Piling are driven until dynamic equation measurements indicate the pile has reached refusal or the required design load indicated in the plan. The service load resistance is monitored in the field using the Mn/DOT
modified ENR formula given in Section 2452.3E of the Mn/DOT Standard Specifications For Construction, 2000 Edition. Designers must calculate the service pile load for the worst load case (Strength I: Final Case 2) and show in the plan, using the Standard Plan Note table for abutments with piling (see Appendix 2-H).

For Strength I: Final Case 2,

\[ P_{serv} = 1959.5 + 949.5 + 297.4 = 3206 \text{ kips} \]

\[ M_{serv \, toe} = -13743.7 - 10462.4 - 1635.7 + 5028.0 = -20,814 \text{ kip-ft} \]

\[ e_{serv \, toe} = \frac{M_{serv \, toe}}{P_{serv}} = \frac{-20,814}{3206} = -6.49 \text{ ft} \]

\[ e_{serv \, NA} = x_{NA} + e_{serv \, toe} = 6.71 - 6.49 = 0.22 \text{ ft} \]

\[ M_{serv \, NA} = P_{serv} \cdot e_{serv \, NA} = 3206 \cdot (0.22) = 705 \text{ kip-ft} \]

\[ \text{Service Pile Load}_{Row \, I} = \frac{3206 + 705 \left( \frac{5.46}{589.4} \right)}{23} \]

\[ = 145.9 \text{ kips/pile} = 73.0 \text{ tons/pile} \]

Also, compute average load factor:

\[ \text{Avg. Load Factor} = \frac{101.3}{73.0} = 1.388 \]

For load table, separate pile load due to factored live load from other factored loads:

Factored \[ P_{LL} = 1.75 \cdot (297.4) = 521 \text{ kips} \]

Factored \[ M_{LL \, NA} = 521 \cdot (6.71 - 5.50) = 630 \text{ kip-ft} \]

Factored Pile Load \[ \text{LL}_{Row \, I} = \frac{521 + 630 \left( \frac{5.46}{589.4} \right)}{23} \]

\[ = 28.5 \text{ kips/pile} = 14.3 \text{ tons/pile} \]
The final results to be shown in the plan are:

<table>
<thead>
<tr>
<th>ABUTMENT</th>
<th>BOTH ABUTMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Computed Pile Load – Tons/Pile</td>
<td></td>
</tr>
<tr>
<td>Factored Dead load + Earth Pressure</td>
<td>87.0</td>
</tr>
<tr>
<td>Factored Live Load</td>
<td>14.3</td>
</tr>
<tr>
<td>Factored Total Load</td>
<td>101.3</td>
</tr>
<tr>
<td>* Design Load</td>
<td>73.0</td>
</tr>
</tbody>
</table>

* \( \frac{101.3}{1.388} = 73.0 \) tons/pile

1.388 is Average Load Factor for Strength I Load Combination

**H. Check Shear in Footing**

General practice is to size the thickness of footings such that shear steel is not required. Try a 48 inch thick footing with a 3 inch step at the toe.

**Determine \( d_v \)**

Based on past design experience assume the bottom mat of steel is \#29 bars spaced at 12 inches (\( A_s = 1.0 \text{ in}^2/\text{ft} \)). The effective shear depth of the section (\( d_v \)) is computed to determine the shear capacity of the footing. The location of the flexural reinforcement is used to determine \( d_v \). The piling has an embedment depth of one foot. Mn/DOT practice is to place the bottom mat of reinforcement directly on top of piling embedded one foot or less. Consequently the cover on the bottom reinforcement is much greater than that on the top mat and will control the computations for \( d_v \). The greater of two equations is used to compute the \( d_v \) value. Note that the 0.72h criterion is not used in this case because the flexural reinforcement location is so high above the bottom of the footing.

Begin by determining the depth of flexural reinforcement:

\[
d_{\text{toe}} = \text{(thickness)} - \text{(pile embedment)} - \left( \frac{d_v}{2} \right) = 51 - 12 - \frac{1.128}{2} = 38.44 \text{ in.}
\]

\[
d_{\text{heel}} = 48 - 12 - \frac{1.128}{2} = 35.44 \text{ in}
\]

The depth of the compression block is:

\[
a = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} = \frac{1.00 \cdot 60}{0.85 \cdot 4 \cdot 12} = 1.47 \text{ in}
\]
The effective shear depth is:

\[ d_{\text{vtoe}} = d - \frac{a}{2} = 38.44 - \frac{1.47}{2} = 37.71 \text{ in} \]

\[ d_{\text{vheel}} = 35.44 - \frac{1.47}{2} = 34.71 \text{ in} \]

It need be no less than 0.9\(d_e\) :

For toe, \(0.9 \cdot d_e = 0.9 \cdot d_{\text{toe}} = 0.9 \cdot 38.44 = 34.90 \text{ in} \)

For heel, \(0.9 \cdot d_e = 0.9 \cdot d_{\text{heel}} = 0.9 \cdot 35.44 = 31.90 \text{ in} \)

Use \(d_{\text{vtoe}} = 37.71 \text{ in}\) and \(d_{\text{vheel}} = 34.71 \text{ in}\)

**Check One-Way Shear in Footing**

The critical section is located \(d_v\) from the face of the abutment. The center line of the Row III piles is 63 inches from the back face of abutment. Therefore, the entire load from the Row III piles contributes to shear on the critical section. Ignore the beneficial effects of the vertical earth loads and footing self weight:

\[ V_{uRow\ III} = \frac{\text{Pile Reaction}}{\text{Pile Spacing}} = \frac{198.1}{8.75} = 22.6 \text{ kips/ft width} \]

The center line of the Row I piles is 33 inches from the front face of abutment. Therefore, only a portion of the load from the Row I piles contributes to shear on the critical section. See Figure 11.4.1.4.
\[ V_{uRowI} = \left( \frac{\text{Pile Fraction Outside Critical Section}}{\text{Pile Reaction}} \right) \left( \frac{\text{Pile Spacing}}{\text{Pile Spacing}} \right) \]

\[ V_{uRowI} = \left( \frac{1.29}{12} \right) \left( \frac{202.6}{7.50} \right) = 2.9 \text{kips/ft width} \]

The shear due to the Row III piles governs.

[5.8.3.3]

There is no shear reinforcement, so the nominal shear capacity of the footing is:

\[ V_n = V_c \]

An upper limit is placed on the maximum nominal shear capacity a section can carry. The maximum design shear for the footing heel is:

\[ V_n = 0.25 \cdot f_c \cdot b_v \cdot d_v = 0.25 \cdot 4.0 \cdot 12.0 \cdot 34.71 = 416.5 \text{ kips} \]

The concrete shear capacity of a section is:

\[ V_c = 0.0316 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v \]

In order to determine \( \beta \), start by calculating the strain \( \varepsilon_x \). For sections without shear reinforcement,

\[ \varepsilon_x = \frac{\left( \frac{M_u}{d_v} + 0.5 N_u + 0.5 (V_u - V_p) \cot \theta - A_{ps} f_{po} \right)}{E_s A_s + E_p A_{ps}} \]

The critical section for moment is the face of the abutment, so

\[ M_u = V_u \cdot \text{Moment Arm} = 22.6 \cdot (5.25) = 118.7 \text{ kip \cdot ft/ft} \]

\[ N_u = V_p = A_{ps} = f_{po} = E_p = A_{ps} = 0 \]

Assume \( \theta = 53.0 \) degrees

Then

\[ \varepsilon_x = \frac{\left[ \frac{118.7 (12)}{34.71} + 0.5 (22.6) \cot 53.0 \right]}{29,000 \cdot 1.00} = 1.71 \times 10^{-3} \]

Next, determine crack spacing parameter \( s_{xe} \)

\[ a_g = 1.5 \text{ inches} \]

\[ s_x = d_v = 34.71 \text{ inches} \]

Then

\[ s_{xe} = s_x \cdot \left( \frac{1.38}{a_g + 0.63} \right) = 34.71 \left( \frac{1.38}{1.50 + 0.63} \right) = 22.49 \text{ in < 80 in } \text{ OK} \]
With $\varepsilon$ and $s_{xe}$ determined, interpolate to find $\beta$ and $\theta$ in LRFD Table 5.3.4.2 – 2.

$\theta = 53.3$ degrees, which is close to the assumed angle
$\beta = 1.38$

For a 1 ft. wide section, substituting values into $V_c$ equation produces:

$$V_c = 0.0316 \cdot 1.38 \cdot \sqrt{4} \cdot 12 \cdot 34.71 = 36.3 \text{ kips}$$

This results in:

$$V_n = V_c = 36.3 \text{ kips} < 416.5 \text{ kips} \quad \text{OK}$$

Including the shear resistance factor, the shear capacity is found to be:

$$V_r = \phi \cdot V_n = 0.90 \cdot 36.3 = 32.7 \text{ kips} > 22.6 \text{ kips} \quad \text{OK}$$

**Check Two-Way Shear in Footing**

Punching of an individual pile through the abutment footing is checked next. The critical section for two-way shear is located at $0.5d_v$ from the perimeter of the pile. The Row I pile at the acute corner is governing case because it has the shortest length of critical section. See Figure 11.4.1.5.

![Figure 11.4.1.5 – Partial Footing Plan](image)

Measured from a CADD drawing, the length of the critical section

$$b_o = 61.4 \text{ in}$$

$$\phi V_n = \phi \left(0.126 \cdot \sqrt{f_c} \cdot b_o d_v\right) = (0.90)(0.126)(\sqrt{4})(61.4)(34.71) = 483.4 \text{ kips}$$

$$V_u = \text{Row I Factored Pile Load} = 202.6 \text{ kips} < 483.4 \text{ kips} \quad \text{OK}$$
The critical section for flexure in the footing is located at the face of the stem for both the top and bottom transverse reinforcement.

1. Top Transverse Reinforcement

Design For Strength Limit State

The design moment for the top transverse bars is found by assuming the heel acts as a cantilever supporting its self weight and the weight of the earth resting on it. In cases where the required reinforcement to resist these loads seems excessive, the moment due to the minimum back pile reaction may be included to decrease the top mat design moment. Use the maximum load factors for DC and EV.

The distributed load associated with the self weight of the footing heel is:

\[ w_{ftg} = \gamma \cdot (\text{thickness}) \cdot (\text{width}) = 0.150 \cdot 4.0 \cdot 1.0 = 0.60 \text{ kips/ft} \]

A heel length of 6.5 feet produces a moment of:

\[ M_{DC} = w_{ftg} \cdot L \cdot \frac{L}{2} = 0.60 \cdot \frac{6.5^2}{2} = 12.7 \text{ kip-ft} \]

The distributed load associated with fill on top of the footing heel is:

\[ w_{EV} = 0.120 \cdot (15.25 + 6.75) \cdot 1.0 = 3.64 \text{ kips/ft} \]

The associated moment in the footing at the stem is:

\[ M_{EV} = 2.64 \cdot \frac{6.5^2}{2} = 55.8 \text{ kip-ft} \]

Combining loads to determine the design moment produces:

\[ M_u = 1.25 \cdot M_{DC} + 1.35 \cdot M_{EV} = 1.25 \cdot 12.7 + 1.35 \cdot 55.8 = 91.2 \text{ kip-ft} \]

Determining the depth of the flexural reinforcement:

\[ d = (\text{thickness}) -(\text{cover}) - \left( \frac{d_b}{2} \right) = 48 - 3 - \frac{1.128}{2} = 44.44 \text{ in.} \]

[5.7.3.2]

Solve for the required area of reinforcing steel:

\[ M_r = \varphi \cdot M_n = \varphi \cdot A_s \cdot f_y \left( d - \frac{A_s \cdot f_y}{2 \cdot 0.85 \cdot f_c' b} \right) \geq M_u \]

Then for \( f_c' = 4.0 \text{ ksi} \) and \( \varphi = 0.90 \),

\[ M_u = 0.90 \cdot A_s \cdot 60 \cdot \left[ d - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 12} \right] \cdot \frac{1}{12} \]

which can be rearranged to:

\[ 3.309 \cdot A_s^2 - 4.5 \cdot d \cdot A_s + M_u = 0 \]
The required area of steel can be found by solving for the smaller root in the quadratic equation.

\[
A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618}
\]

The required area of steel is 0.46 in\(^2\)/ft. Try #22 bars at 12 inches (A\(_s\) = 0.60 in\(^2\)/ft).

[5.7.3.3.1] **Check Maximum Reinforcement**

No more than 42 percent of the flexural cross section can be in compression at the strength limit state.

For #22 bars, \(d = 48 - 3 \cdot \frac{0.875}{2} = 44.56\) in.

\[
c = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = \frac{0.60 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 1.04\text{ in.}
\]

The fraction of the section in compression is:

\[
\frac{c}{d_e} = \frac{1.04}{44.56} = 0.023 < 0.42\text{ OK}
\]

[5.7.3.3.2] **Check Minimum Reinforcement**

The minimum reinforcement check has two parts. The flexural reinforcement needs to be able to carry a moment 20 percent larger than the cracking moment of the cross section. If this criteria is not satisfied, the amount of reinforcement needs to be increased to carry the lesser of 1.20 times the cracking moment or 1.33 times the original design moment.

The rupture stress of concrete in flexure is:

\[
f_r = 0.24 \cdot \sqrt{f'c} = 0.24 \cdot \sqrt{4} = 0.48\text{ ksi}
\]

The gross moment of inertia is:

\[
I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (48)^3 = 110,600\text{ in}^4
\]

The distance from the centroidal axis to the tension face is:

\(y_t = 24.00\) in

Combining these parameters leads to a cracking moment of:

\[
M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.48 \cdot 110,600}{24.00 \cdot (12)} = 184.3\text{ kip-ft}
\]

With a 20 percent increase, the required capacity is:

\(1.2 M_{cr} = 221.2\text{ kip-ft}\)
The capacity of the #22 bars at a 12 inch spacing is:

\[ M_r = \varphi \cdot A_s \cdot f_y \cdot \left( d - \frac{a}{2} \right) \]

\[ M_r = 0.9 \cdot 0.60 \cdot 60 \left( 44.56 - \frac{1.04 \cdot 0.85}{2} \right) \left( \frac{1}{12} \right) = 119.1 \text{ kip - ft} \]

Which is less than the 221.2 kip - ft capacity required.

The strength design moment of 91.2 kip-ft is less than half of the \( 1.2 \cdot M_{cr} \) moment. Provide reinforcement capable of resisting:

\[ 1.33 \cdot M_u = 1.33 \cdot 91.2 = 121.8 \text{ kip - ft} \]

The #22 bars at 12 inches, with a capacity of 119.1 kip – ft, are within 3% of the required capacity. Consider the design adequate.

2. **Bottom Transverse Reinforcement**

**Design For Strength Limit State**

Although the toe has a greater thickness than the heel, for simplicity assume a constant thickness of 48 inches. Then the design moment for the bottom mat is the largest of the moments due to the maximum pile reactions for the Row I or Row III piles.

For the Row I piles:

\[ M_{u_{RowI}} = \frac{\text{Pile Reaction}}{\text{Pile Spacing}} \cdot \text{(Moment Arm)} \]

\[ = \left( \frac{202.6}{7.5} \right) \cdot 4.00 \cdot 1.25 = 74.3 \text{ kip – ft/ft width} \]

For the Row III piles, subtract off the moment due to earth on the heel when calculating the factored moment:

\[ M_{u_{RowIII}} = \frac{\text{Pile Reaction}}{\text{Pile Spacing}} \cdot \text{(Moment Arm)} - \varphi M_{ev} \]

\[ = \left( \frac{198.1}{8.75} \right) \cdot 6.50 \cdot 1.25 - 0.90 \cdot (55.8) = 68.6 \text{ kip – ft/ft width} \]

The Row I moment governs. \( M_{udes} = 74.3 \text{ kip - ft/ft width} \)

Assuming #29 bars, the depth of the bottom flexural reinforcement is:

\[ d = (\text{thickness}) - (\text{pile embedment}) - \left( \frac{d_b}{2} \right) = 48 - 12 - \frac{1.128}{2} = 35.44 \text{ in.} \]
Solve once again with:
\[ A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618} \]

The required area of steel is 0.47 in²/ft. Try #22 bars at 12 inches with standard hooks (A_s = 0.60 in²/ft).

[5.7.3.3.2] Check Minimum Reinforcement
The minimum reinforcement check for the bottom of the footing has the same steps as the other elements.

The gross moment of inertia is:
\[ I_g = \frac{1}{12} b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (48)^3 = 110,600 \text{ in}^4 \]

The distance from the centroidal axis to the tension face is:
\[ y_t = 24.00 \text{ in} \]

Combining these parameters and using the rupture stress computed earlier leads to a cracking moment of:
\[ M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.48 \cdot 110,600}{24.00 \cdot (12)} = 184.3 \text{ ft-kip} \]

With a 20 percent increase, the required capacity is:
\[ 1.2 M_{cr} = 221.2 \text{ kip-ft} \]

For #22 bars, \[ d = 48 - 12 - \frac{0.875}{2} = 35.56 \text{ in} \]

The capacity of the #22 bars at a 12 inch spacing is:
\[ M_r = \varphi \cdot A_s \cdot f_y \cdot \left( d - \frac{a}{2} \right) \]
\[ M_r = 0.9 \cdot 0.60 \cdot 60 \cdot \left( 35.56 - \frac{1.04 \cdot 1.85}{2} \right) \cdot \left( \frac{1}{12} \right) \]
\[ = 94.8 \text{ kip-ft} < 221.2 \text{ kip-ft} \text{ NO GOOD} \]

Therefore, we must provide reinforcement capable of resisting:
\[ 1.33 \cdot M_u = 1.33 \cdot 74.3 \]
\[ = 98.4 \text{ kip-ft} > 94.8 \text{ kip-ft by 4.2% NO GOOD} \]

Revise reinforcement to #25 bars at 12 inches (A_s = 0.79 in²/ft). Then
\[ M_r = 124.1 \text{ kip-ft} > = 98.8 \text{ kip-ft OK} \]
Check Maximum Reinforcement

For #25 bars, 

\[ d = 48 - 12 - \frac{1.00}{2} = 35.50 \text{ in} \]

\[ c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{0.79 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 1.37 \text{ in} \]

The fraction of the section in compression is:

\[ \frac{c}{d_e} = \frac{1.37}{35.50} = 0.039 < 0.42 \quad \text{OK} \]

Provide #25 bars at 12 inches \((A_s = 0.79 \text{ in}^2/\text{ft})\).

3. Longitudinal Reinforcement

Design For Strength Limit State

For longitudinal bars, design for uniform load due to all vertical loads spread equally over the length of the footing. Assume the footing acts as a continuous beam over pile supports. Use the longest pile spacing for design span.

Then based on the maximum vertical load from Table 11.4.1.5:

\[ w_u = \frac{4252}{55.33} = 76.8 \text{ kips/ft} \]

\[ M_u = \frac{w_u L^2}{10} = \frac{76.8 \cdot (8.75)^2}{10} = 588.0 \text{ kip} \cdot \text{ft} \]

Assume #19 bars, which is the smallest size used by Mn/DOT in footings:

\[ d = 48 - 12 - \frac{1.00 \cdot 0.75}{2} = 34.63 \text{ in} \]

Solve for required area of reinforcement:

\[ M_r = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \left( d - \frac{A_s \cdot f_y}{2 \cdot 0.85 \cdot f'_c \cdot b} \right) \geq M_u \]

Then \( 588.0 = 0.90 \cdot A_s \cdot 60 \left( 34.63 - \frac{A_s \cdot 60}{2 \cdot 0.85 \cdot 4 \cdot 180} \right) \cdot \frac{1}{12} \)

Rearrange and get \( 0.2206 \cdot A_s^2 - 155.84 \cdot A_s + 588.0 = 0 \)

Solving, minimum \( A_s = 3.79 \text{ in}^2 \)

Try 15-#19 bars at a 12 inch spacing. \((A_s = 6.60 \text{ in}^2)\)
Check Minimum Reinforcement
By inspection, $M_r > 1.33 \cdot M_u$  OK

Check Maximum Reinforcement
\[
c = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = \frac{6.60 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 180} = 0.76 \text{ in}
\]
\[
c = \frac{0.76}{34.63} = 0.0219 < 0.42 \text{ OK}
\]

Provide 15-#19 bars at 12 inches ($A_s = 6.60 \text{ in}^2$) for the footing longitudinal reinforcement.

**J. Flexural Design of the Stem**

The moments associated with the eccentricity of vertical loads are minimal and are therefore ignored. Use a one-foot wide design strip. The stem design is governed by the horizontal earth pressure and live load surcharge loading during construction.

**[3.11.5.5]**

**Horizontal Earth Pressure**
\[
p_{\text{top}} = 0.0 \text{ ksf}
\]
\[
p_{\text{bottom}} = 0.033 \cdot 22.25 = 0.734 \text{ ksf}
\]
The resultant force applied to the stem is:
\[
P_{\text{EH}} = 0.5 \cdot (0.734) \cdot (22.25) \cdot (1.00) = 8.17 \text{ kips}
\]
The height of the resultant above the footing is:
\[
x_{\text{EH}} = 0.33 \cdot 22.25 = 7.34 \text{ ft}
\]
The moment at the base of the stem is:
\[
M_{\text{EH}} = P_{\text{EH}} \cdot x_{\text{EH}} = 8.17 \cdot 7.34 = 60.0 \text{ kp-ft}
\]

**[Table 3.11.6.4-1]**

**Live Load Surcharge**
For walls over 20 feet in height, $h_{eq}$ is 2 feet.

The resultant force applied to the stem is:
\[
P_{\text{LS}} = 0.033 \cdot (2.00) \cdot (22.25) \cdot (1.00) = 1.47 \text{ kips}
\]
The height of the resultant force above the footing is:
\[
x_{\text{LS}} = \frac{22.25}{2} = 11.13 \text{ ft}
\]
The moment at the base of the stem is:
\[
M_{\text{LS}} = P_{\text{LS}} \cdot x_{\text{LS}} = 1.47 \cdot 11.13 = 16.4 \text{ kip-ft}
\]
**Design Moments**

The design factored moment is:

\[ M_u = 1.5 \cdot M_{EH} + 1.75 \cdot M_{LS} = 1.50 \cdot 60.0 + 1.75 \cdot 16.4 = 118.7 \text{ kip - ft} \]

The design service moment is:

\[ M_{\text{service}} = 1.0 \cdot M_{EH} + 1.0 \cdot M_{LS} = 1.0 \cdot 60.0 + 1.0 \cdot 16.4 = 76.4 \text{ kip - ft} \]

**Figure 11.4.1.4 - Load Diagram for Stem Design**

**[5.7.2.2]** **Investigate the Strength Limit State**

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment.

\[ M_u = 118.7 \text{ kip - ft} \]

Initially, assume that #19 bars are used for flexural reinforcement to compute the “d” dimension:

\[ d = (\text{thickness}) - (\text{cover}) - \left( \frac{d_b}{2} \right) = 54 - 2 - \frac{0.75}{2} = 51.63 \text{ in} \]

Using

\[ A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618} \]
The required area of steel is 0.51 in$^2$/ft. Try #16 bars at 6 inches ($A_s=0.62$ in$^2$/ft, $d=51.69$ in).

**Crack Control**

Check crack control equations to ensure that the primary reinforcement is well distributed.

The transformed area of the reinforcement is:

\[
96.46 \times 0.62 = 59.64 \text{ in}^2
\]

Figure 11.4.1.5

Determine the location of the neutral axis:

\[
\frac{1}{2} \cdot b x^2 = n \cdot A_s (d - x)
\]

\[
\frac{1}{2} \cdot (12) \cdot x^2 = 4.96 \times (51.69 - x) \quad \text{solving, } x = 6.14 \text{ inches}
\]

\[
j \cdot d = d - \frac{x}{3} = 51.69 - \frac{6.14}{3} = 49.64 \text{ in}
\]

Actual $f_s = \frac{M}{A_s \cdot j \cdot d} = \frac{76.4 \times 12}{0.62 \times (49.64)} = 29.8$ ksi
For \( z = 170 \text{kips/in} \), \( d_c = 2.313 \text{ inches} \), #16 bars at 6 inches;

\[
A = \frac{2 \cdot (d_c) \cdot b}{N} = \frac{2 \cdot (2.313) \cdot 12}{2} = 27.8 \text{ in}^2
\]

Permitted \( f_s = \frac{z}{3d_c \cdot A} = \frac{170}{\sqrt[3]{2.313 \cdot 27.8}} = 42.4 > 0.6 \cdot f_y = 36 \text{ ksi} \)

Actual \( f_s = 29.8 \text{ ksi} < \text{Permitted } f_s = 36.0 \text{ ksi} \quad \text{OK} \)

[5.7.3.3.1]

**Check Maximum Reinforcement**

No more than 42 percent of the flexural cross section can be in compression at the strength limit state. With \( A_s = 0.62 \text{ in}^2/\text{ft} \) and \( d = 51.69 \text{ inches} \), the depth of the section in compression is:

\[
c = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b_1 \cdot b} = \frac{0.62 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 1.07 \text{ in}
\]

The fraction of the section in compression is:

\[
\frac{c}{d} = \frac{1.07}{51.69} = 0.021 < 0.42 \quad \text{OK}
\]

[5.7.3.3.2]

**Check Minimum Reinforcement**

The gross moment of inertia is:

\[
I_g = \frac{1}{12} b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (54)^3 = 157,500 \text{ in}^4
\]

The distance from the centroidal axis to the tension face is:

\( y_t = 27.00 \text{ in} \)

Combining these parameters and using the rupture stress computed earlier leads to a cracking moment of:

\[
M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.48 \cdot 157,500}{27.00 \cdot (12)} = 233.3 \text{ kip-ft}
\]

With a 20 percent increase, the required capacity is:

\( 1.2 \ M_{cr} = 280.0 \text{ kip-ft} \)

The capacity of the #16 bars at a 6 inch spacing is:

\[
M_r = \phi \cdot A_s \cdot f_y \cdot \left( d - \frac{a}{2} \right)
\]

\[
M_r = 0.9 \cdot 0.62 \cdot 60 \left( 51.69 - \frac{1.07 \cdot 1.85}{2} \right) \left( \frac{1}{12} \right) = 142.9 \text{ kip-ft}
\]
The strength design moment of 118.7 kip-ft is less than half of the 1.2·Mcritical moment. Provide reinforcement capable of resisting:

\[ 1.33 \cdot M_u = 1.33 \cdot 118.7 = 157.9 \text{ kip-ft} \]

With #19 bars (d = 51.63) the required area of steel is 0.69 in²/ft.

Provide #19 bars at 6 inches (As = 0.88 in²/ft) for vertical back face reinforcement.

Shrinkage and Temperature Reinforcement

Mn/DOT practice for members over 48 inches thick is to use #19 bars spaced at 12 inches.

Use #19 bars at 12 inches (As = 0.44 in²/ft) on each face, for horizontal reinforcement and #19 bars at 12 inches for vertical front face reinforcement.

### K. Splice Length

**5.11.5.3**

**5.11.2**

Calculate the tension lap length for the stem reinforcing. For epoxy coated #19 bars the basic development length \( l_{db} \) is the greater of:

\[ l_{db} = \frac{1.25 \cdot A_b \cdot f_y}{\sqrt{f'_c}} = \frac{1.25 \cdot 0.44 \cdot 60}{\sqrt{4.0}} = 16.5 \text{ in.} \]

or

\[ l_{db} = 0.4 \cdot d_b \cdot f_y = 0.4 \cdot 0.75 \cdot 60 = 18.0 \text{ in.} \]

GOVERNS

The modification factors to the development length are:

- 1.5 for epoxy coated bars with cover less than three bar diameters (2.25 in).
- 0.8 for bars with spacing \( \geq 6 \) inches and cover \( \geq 3 \) inches in direction of spacing. (Note that cover for the end bars is < 3 inches, but the wall is long, so cover will have negligible effect.)

Then the development length \( l_d \) is:

\[ l_d = 18.0 \cdot 1.5 \cdot 0.8 = 21.6 \text{ in.} \]

Referring to LRFD Table 5.11.5.3.1-1, with 100 percent of the steel spliced and less than twice the necessary amount of steel provided, a Class C splice should be provided.

**5.11.5.3.1**

The required lap length \( l_{spl} \) is:
\[ \ell_{spl} = 1.70 \cdot \ell_d = 1.70 \cdot 21.6 = 36.7 \text{ in} \]

Use a tension lap length of 37 inches.

**L. Flexural Design of the Backwall (parapet)**

The required vertical reinforcement in the backwall (parapet) is sized to carry the moment at the bottom of the backwall. The design is performed on a one-foot wide strip of wall. The backwall design is governed by the horizontal earth pressure and live load surcharge loading during construction.

**Horizontal Earth Pressure**

\[ p_{top} = 0.0 \text{ ksf} \]

\[ p_{bottom} = 0.033 \cdot 6.75 = 0.223 \text{ ksf} \]

The resultant force applied to the backwall is: \[ P_{EH} = 0.5 \cdot (0.223) \cdot (6.75) \cdot (1.00) = 0.75 \text{ kips} \]

The height of the resultant above the bottom of the backwall is: \[ x_{EH} = 0.33 \cdot (6.75) = 2.25 \text{ ft} \]

The moment at the bottom of the backwall is: \[ M_{EH} = P_{EH} \cdot x_{EH} = 0.75 \cdot 2.25 = 1.69 \text{ kip \cdot ft} \]

**[Table 3.11.6.4-1]**

**Live Load Surcharge**

Interpolate between the values provided in the table to arrive at the required equivalent height of surcharge to use for the design of the backwall.

\[ h_{eq} = \left( \frac{6.75 - 5}{10.0 - 5} \right) \cdot (3 - 4) + 4 = 3.65 \text{ ft} \]

The resultant force applied to the backwall is: \[ P_{LS} = 0.033 \cdot (3.65) \cdot (6.75) \cdot (1.00) = 0.81 \text{ kips} \]

The height of the resultant force above the bottom of the backwall is: \[ x_{LS} = \frac{6.75}{2} = 3.38 \text{ ft} \]

Moment at the bottom of the backwall is: \[ M_{LS} = P_{LS} \cdot x_{LS} = 0.81 \cdot 3.38 = 2.74 \text{ kip \cdot ft} \]
Figure 11.4.1.6 - Load Diagram for Backwall Design

Design Moments
Combining the load factors for the EH and LS load components with the flexural design forces at the bottom of the backwall produces the following design forces.

\[ M_U = 1.5M_{EH} + 1.75M_{LS} = 1.5(1.69) + 1.75(2.74) = 7.33 \text{ kip - ft} \]

\[ M_{SERVICE} = M_{EH} + M_{LS} = 1.69 + 2.74 = 4.43 \text{ kip - ft} \]

[5.7.2.2] Investigate the Strength Limit State
Determine the area of back-face flexural reinforcement necessary to satisfy the design moment.

Once again, use
\[ A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 13.236 \cdot M_u}}{6.618} \]

Initially, assume that #19 bars are used for flexural reinforcement to compute the “d” dimension:
Solving the equation, the required area of steel is 0.10 in\(^2\)/ft.

In no case should reinforcement be less than #16 bars at a 12 inch spacing. The area of steel for #16 bars at 12 inches is 0.31 in\(^2\)/ft.

Continue the backwall flexural checks using #16 bars at 12 inches. The actual "d" for this reinforcement layout is:

\[
d = 18 - 2 - \frac{0.625}{2} = 15.69 \text{ in.}
\]

**Check Crack Control**

Check crack control equations to ensure that the primary reinforcement is well distributed. Design for a z value of 170 kip/in.

To check if steel stresses are acceptable, determine the cracked section properties with the trial reinforcement.

Compute the modular ratio for 4.0 ksi concrete:

\[
n = \frac{E_s}{E_c} = \frac{29,000}{33,000 \cdot (0.145)^{1.5} \cdot \sqrt[4]{4}} = 7.96
\]

Use 8

The transformed area of the reinforcement is:

\[
n \cdot A_s = 8 \cdot 0.31 = 2.48 \text{ in}^2
\]

Determine the location of the neutral axis:

\[
\frac{1}{2} \cdot bx^2 = n \cdot A_s (d_s - x)
\]

\[
\frac{1}{2} \cdot (12) \cdot x^2 = 2.48 (15.69 - x) \quad \text{solving, } x = 2.35 \text{ inches}
\]

\[
j \cdot d = d - \frac{x}{3} = 15.69 - \frac{2.35}{3} = 14.91 \text{ in}
\]

Actual \(f_s = \frac{M}{A_s \cdot j \cdot d} = \frac{4.43 \cdot 12}{0.31 \cdot (14.91)} = 11.5 \text{ ksi}\)

For \(z = 170 \text{ kips/in}, d_c = 2.31 \text{ inches, #16 bars at 12 inches;}

\[
A = \frac{2 \cdot (d_c) \cdot b}{N} = \frac{2 \cdot (2.31) \cdot 12}{1} = 55.4 \text{ in}^2
\]
Permitted $f_s = \frac{z}{\frac{3}{2}d_c \cdot A} = \frac{170}{\frac{3}{2} \cdot 2.31 \cdot 55.4} = 33.7 \text{ ksi} < 0.6 \cdot f_y = 36.0 \text{ ksi}$

Actual $f_s = 11.5 \text{ ksi} < \text{Permitted } f_s = 33.7 \text{ ksi}$  OK

**[5.7.3.3.1]**

**Check Maximum Reinforcement**

No more than 42 percent of the flexural cross section can be in compression at the strength limit state. With $A_s = 0.31 \text{ in}^2/\text{ft}$ and $d = 15.69 \text{ inches}$, the depth of the section in compression is:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_{c,b}} \cdot b = \frac{0.31 \cdot 60}{0.85 \cdot 4 \cdot 0.85 \cdot 12} = 0.54 \text{ in.}$$

The fraction of the section in compression is:

$$\frac{c}{d} = \frac{0.54}{15.69} = 0.034 < 0.42 \text{ OK}$$

**[5.7.3.3.2]**

**Check Minimum Reinforcement**

The gross moment of inertia is:

$$I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 12 \cdot (18)^3 = 5,832 \text{ in}^4$$

The distance from the centroidal axis to the tension face is:

$$y_t = 9.00 \text{ in}$$

Combining these parameters leads to a cracking moment of:

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.48 \cdot 5,832}{9.00 \cdot (12)} = 25.9 \text{ kip-ft}$$

And with the 20 percent increase, the required capacity is:

$$1.2 \cdot M_{cr} = 31.1 \text{ kip-ft}$$

The capacity of the #16 bars at a 12 inch spacing is:

$$M_r = \varphi \cdot A_s \cdot f_y \cdot \left( d - \frac{a}{2} \right)$$

$$M_r = 0.9 \cdot 0.31 \cdot 60 \cdot \left( 15.69 - \frac{0.54 \cdot 0.85}{2} \right) \cdot \left( \frac{1}{12} \right) = 21.6 < 31.1 \text{ kip-ft}$$

The required steel area due to flexure is 0.10 in$^2$/ft. The minimum steel permitted (#16 bars at 12 inches) has an area of 0.31 in$^2$/ft. Consequently, the minimum steel provides more than 1.33 of the required steel.

Use #16 bars at 12 inches for vertical back face reinforcement.
Shrinkage and Temperature Reinforcement

To distribute and limit the size of cracks associated with concrete shrinkage and with temperature changes, a modest amount of reinforcement is provided transverse to the primary reinforcement.

The total area of required transverse reinforcement to be distributed to both faces is:

\[ A_s \geq 0.11 \cdot \left( \frac{A_g}{f_y} \right) = 0.11 \cdot \left( \frac{18 \cdot 12}{60} \right) = 0.40 \text{ in}^2/\text{ft} \]

Provide horizontal #16 bars at 12 inches to both faces, \( A_s = 0.31 \text{ in}^2/\text{ft} \)

The final reinforcing layout, is presented in Figure 11.4.1.7.
This design example illustrates the design of a cantilever retaining wall supported on a spread footing. The wall supports 13'-0" of fill, an "F" rail, and has no counterforts. After assembling the load components and design loads, the global behavior of the retaining wall is evaluated. This includes: an eccentricity or overturning check, a bearing stress check, and a sliding check, after which, the wall section is designed. The design example concludes with shear and flexural checks of the footing.

A preliminary cross-section of the wall is shown in Figure 11.4.2.1. Choose a footing width between 70 and 75 percent of the stem height and a footing thickness between 10 and 15 percent of the stem height. Choose a toe projection equal to approximately 30 percent of the footing width.
Material and design parameters used in this example are:

Soil:
- The soil is noncohesive.
- Unit Weight of fill, $w_s = 0.120 \text{ kcf}$
- Ultimate Soil Capacity, $q_n = 4.5 \text{ tsf}$
- Internal friction angle of foundation soil, $\phi_f = 30^\circ$

Concrete:
- Strength at 28 Days, $f'_c = 4.0 \text{ ksi}$
- Unit Weight, $w_c = 0.150 \text{ kcf}$

Reinforcement:
- Yield Strength, $f_y = 60 \text{ ksi}$
- Modulus of Elasticity, $E_s = 29,000 \text{ ksi}$

Barrier ("F" rail) Weight = 0.439 k/ft

The $q_n$ values are provided to designers in the foundation recommendation report. The report is based on standard penetration test (SPT) data.

$$\phi = 0.45 \text{ for soil resistance}$$

$$q_r = \phi \cdot q_n = 0.45 \cdot (4.5) = 2.03 \text{ tsf}$$

The design of the retaining wall is performed on a strip 1'-0" wide. Figure 11.4.2.2 shows a section of the retaining wall. Soil and concrete elements are broken into rectangles or triangles. Each rectangle or triangle is labeled with two numbers. The first number is the unfactored weight of the region and the second number (the number in parenthesis) is the horizontal distance "x" from the toe of the footing to the center of the region.

For level fill applications, the equivalent-fluid method of LRFD Section 3.11.5.5 can be used to determine the magnitude of active earth pressure. For dense sand backfills, equivalent fluids with a unit weight between 0.030 and 0.035 kcf are appropriate. Mn/DOT practice is to use 0.033 kcf.

$$\gamma_{eq} = 0.033 \text{ kcf}$$
For a 1 foot wide design strip, the horizontal earth pressure is:

\[ P_{EH\text{top}} = (0.033) (0) (1) = 0 \text{ kips/ft} \]

\[ P_{EH\text{bottom}} = (0.033) (14.50) (1) = 0.479 \text{ kips/ft} \]

\[ P_{EH} = \frac{1}{2} (0.479) (14.50) = 3.47 \text{ kips} \]

The earth pressure resultant is applied at:

\[ y_{EH} = \frac{S}{3} = \frac{14.50}{3} = 4.83 \text{ ft above bottom of footing} \]

See Figure 11.3.2.2 for application of the earth pressure load.

**D. Live Load Surcharge (LS) [3.11.6.4]**

The horizontal pressure \( p_{LS} \) due to live load surcharge is:

\[ p_{LS} = \gamma_{eq} \cdot h_{eq} \]

From Table 3.11.6.4-2, use \( h_{eq} = 2.0 \text{ ft} \) based on a distance from wall backface to edge of traffic \( \geq 1 \text{ ft} \).

\[ p_{LS} = (0.033) (2.0) = 0.066 \text{ ksf} \]

Horizontal Component of LS:

\[ P_{LS} = (0.066) (14.50) = 0.96 \text{ kips} \]

The live load surcharge resultant is applied at:

\[ y_{LS} = \frac{S}{2} = \frac{14.50}{2} = 7.25 \text{ ft above bottom of footing} \]

See Figure 11.4.2.2 for application of the live load surcharge.
E. Barrier Loading (CT) [A13.2]

The vehicle collision load and Extreme Event II limit state will be considered only when checking overturning, bearing, and sliding of the wall. A yield line analysis shows that the F-rail reinforcement is adequate to resist the vehicle collision load. (See F-Rail Design Example in Section 13.3.1 of this manual.) The retaining wall reinforcement will be the same or greater than the F-rail reinforcement. Therefore, by inspection, it will also be adequate to resist the collision load.

From Section 13.3.1E, the length of the end region $L_{ce}$ for barrier load distribution is:

$$L_{ce} = 5.0 \text{ ft (for F-rail)}$$

Assume that the load distributes within the wall with a 45° slope and that one end of the retaining wall is vertical (next to the end of the wall or
adjacent to a vertical joint in the wall.) The elevation view presented in Figure 11.4.2.3 shows the assumed effective length of the retaining wall.

The horizontal vehicle collision force applied to the barrier is 54 kips.

\[ P_{CT} = \frac{54}{5.0 + 2.67 + 13 + 1.50} = 2.44 \text{ kip/ft width} \]

\[ \gamma_{CT} = 2.67 = 13.0 = 1.50 = 17.17 \text{ ft above bottom of footing} \]

Per LRFD Article 13.6.2, this loading is already factored and is to be considered at the extreme event limit state.
For typical retaining walls use:  
\[ \eta_D = 1, \quad \eta_R = 1, \quad \eta_I = 1 \]

Table 11.4.2.1 summarizes the load combinations used for design of the wall. Strength Ia and Extreme Event IIa, both used to check sliding and overturning, have minimum load factors for the vertical loads and maximum load factors for the horizontal loads. Strength Ib and Extreme Event IIb are used to check bearing and have maximum load factors for both vertical and horizontal loads. Note that live load surcharge (LS) and horizontal earth (EH) are not included in Extreme Event IIa or IIb. The vehicle collision load (CT) is an instantaneous load applied in the same direction as LS and EH. Because of its instantaneous nature, it has the effect of unloading LS and EH. Therefore, the three loads are not additive and only CT is included in the Extreme Event load combinations.

The service limit state is used for the crack control check.

<table>
<thead>
<tr>
<th>Load Comb.</th>
<th>( \gamma_{DC} )</th>
<th>( \gamma_{EV} )</th>
<th>( \gamma_{LS} )</th>
<th>( \gamma_{EH} )</th>
<th>( \gamma_{CT} )</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength Ia</td>
<td>0.90</td>
<td>1.00</td>
<td>1.75</td>
<td>1.50</td>
<td>-</td>
<td>Sliding, overturning</td>
</tr>
<tr>
<td>Strength Ib</td>
<td>1.25</td>
<td>1.35</td>
<td>1.75</td>
<td>1.50</td>
<td>-</td>
<td>Bearing, Wall Strength</td>
</tr>
<tr>
<td>Extreme IIa</td>
<td>0.90</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
<td>Sliding, overturning</td>
</tr>
<tr>
<td>Extreme IIb</td>
<td>1.25</td>
<td>1.35</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
<td>Bearing</td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>-</td>
<td>Wall Crack Control</td>
</tr>
</tbody>
</table>

The vertical loads and lever arms to the toe of the footing for the earth and concrete dead loads in Figure 11.4.2.2 are summarized in Table 11.4.2.2. Also presented are the corresponding moments about the toe. \( P_s \) pertains to the stem and barrier. \( P_f \) pertains to the footing, and EV summarizes the vertical earth loads on both the toe and heel.
Table 11.4.2.2
Unfactored Vertical Loads and Moments about Toe

<table>
<thead>
<tr>
<th>Vertical Load</th>
<th>V (kips)</th>
<th>Moment Arm About Toe x (ft)</th>
<th>Moment About Toe (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ps1 (barrier)</td>
<td>0.44</td>
<td>3.32</td>
<td>1.46</td>
</tr>
<tr>
<td>Ps2 (rectangle)</td>
<td>2.93</td>
<td>3.50</td>
<td>10.26</td>
</tr>
<tr>
<td>Ps3 (triangle)</td>
<td>0.53</td>
<td>4.43</td>
<td>2.35</td>
</tr>
<tr>
<td>P (resultant)</td>
<td>3.90</td>
<td>3.61</td>
<td>14.07</td>
</tr>
<tr>
<td>P1 (1'-7/2&quot; thick)</td>
<td>0.74</td>
<td>1.52</td>
<td>1.12</td>
</tr>
<tr>
<td>P2 (1'-6&quot; thick)</td>
<td>1.40</td>
<td>6.15</td>
<td>8.61</td>
</tr>
<tr>
<td>P (resultant)</td>
<td>2.14</td>
<td>4.55</td>
<td>9.73</td>
</tr>
<tr>
<td>EV1 (toe)</td>
<td>0.95</td>
<td>1.38</td>
<td>1.31</td>
</tr>
<tr>
<td>EV2 (rectangle)</td>
<td>6.96</td>
<td>7.02</td>
<td>48.86</td>
</tr>
<tr>
<td>EV3 (triangle)</td>
<td>0.42</td>
<td>4.61</td>
<td>1.94</td>
</tr>
<tr>
<td>EV (resultant)</td>
<td>8.33</td>
<td>6.26</td>
<td>52.11</td>
</tr>
</tbody>
</table>

Table 11.4.2.3 contains similar data for the horizontal loads presented in Figures 11.4.2.2 and 11.4.2.3.

Table 11.4.2.3
Unfactored Horizontal Loads and Overturning Moments

<table>
<thead>
<tr>
<th>Horizontal Load</th>
<th>H (kip)</th>
<th>Moment Arm To Bottom of Footing y (feet)</th>
<th>Moment (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PEH</td>
<td>3.47</td>
<td>4.83</td>
<td>16.76</td>
</tr>
<tr>
<td>PLS</td>
<td>0.96</td>
<td>7.25</td>
<td>6.96</td>
</tr>
<tr>
<td>PCT</td>
<td>2.44</td>
<td>17.17</td>
<td>41.89</td>
</tr>
</tbody>
</table>

Calculate loads and moments acting on the retaining wall for the different load combinations. An example calculation for the Strength Ia load combination is shown below. Results for other load combinations are shown in Table 11.4.2.4.

Strength Ia: \( \eta_D \cdot \eta_R \cdot \eta_I (0.90 \, DC + 1.0 \, EV + 1.75 \, LS + 1.50 \, EH) \)

\[ V = 1.0 \left[ 0.90 \left( 3.90 + 2.14 \right) + 1.0 \left( 8.33 \right) \right] \]

\[ = 13.77 \text{ kips} \]
\[ M_V = 1.0 \times [0.90(14.07 + 9.73) - 1.0(52.11)] = 73.53 \text{ kip-ft} \]
\[ H = 1.0 \times (1.75 \times 0.96 + 1.50 \times 3.47) = 6.89 \text{ kips} \]
\[ M_H = 1.0 \times (1.75 \times 6.96 + 1.50 \times 16.76) = 37.32 \text{ kip-ft} \]

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Vertical Load V (kips)</th>
<th>Moment MV (kip-ft)</th>
<th>Horizontal Load H (kips)</th>
<th>Moment MH (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength Ia</td>
<td>13.77</td>
<td>73.53</td>
<td>6.89</td>
<td>37.32</td>
</tr>
<tr>
<td>Strength Ib</td>
<td>18.80</td>
<td>100.10</td>
<td>6.89</td>
<td>37.32</td>
</tr>
<tr>
<td>Extreme Ev. Iia</td>
<td>13.77</td>
<td>73.53</td>
<td>2.44</td>
<td>41.89</td>
</tr>
<tr>
<td>Extreme Ev. Iib</td>
<td>18.80</td>
<td>100.10</td>
<td>2.44</td>
<td>41.89</td>
</tr>
<tr>
<td>Service I</td>
<td>14.37</td>
<td>75.91</td>
<td>4.43</td>
<td>23.72</td>
</tr>
</tbody>
</table>

**I. Check Overturning**

The width of footing dimension is designated as “d” in the Bridge Standard Plans for retaining walls. The LRFD Specifications designate the width of the footing as “B”. For this example, the foundation rests on soil.

\[ e_{\text{max}} = \frac{B}{4} = \frac{d}{4} = \frac{9.25}{4} = 2.31 \text{ ft} \]

Using the following relationships, compare the actual eccentricity \(e\) to \(e_{\text{max}}\):

\[ x_r = \frac{M_V - M_H}{V}, \quad \text{Actual } e = \frac{d}{2} - x_r \]

where \(x_r\) = location of resultant from the toe

For Strength Ia:
\[ x_r = \frac{73.53 - 37.32}{13.77} = 2.63 \text{ ft} \]
\[ \text{Actual } e = \frac{9.25}{2} - 2.63 = 2.00 \text{ ft} < 2.31 \text{ ft} \quad \text{OK} \]

For Extreme Event IIa:
\[ x_r = \frac{73.53 - 41.89}{13.77} = 2.30 \text{ ft} \]
\[ \text{Actual } e = \frac{9.25}{2} - 2.30 = 2.33 \text{ ft} > 2.31 \text{ ft by 0.87%} \quad \text{SAY OK} \]
The footing size is satisfactory for overturning.

**J. Check Bearing**  
**[11.6.3.2]**

Determine if the bearing resistance \( q_r = 2.03 \text{ tsf} \) is adequate for the calculated bearing pressure \( \sigma_{V} \).

\[
\sigma_{V} = \frac{\Sigma V}{(d - 2e)}
\]

where \( \Sigma V \) = summation of vertical forces per unit length of wall

For Strength Ib:

\[
x_r = \frac{M_v - M_h}{V} = \frac{100.10 - 37.32}{18.80} = 3.34 \text{ ft} \\
e = \frac{d}{2} - x_r = \frac{9.25}{2} - 3.34 = 1.29 \text{ ft} \\
\sigma_{V} = \frac{18.80}{9.25 - 2 \cdot (1.29)} \cdot \left(\frac{1}{2}\right) = 1.41 \text{ tsf} < 2.03 \text{ tsf} \quad \text{OK}
\]

For Extreme Event IIb:

\[
x_r = \frac{100.10 - 41.89}{18.80} = 3.10 \text{ ft} \\
e = \frac{9.25}{2} - 3.10 = 1.53 \text{ ft} \\
\sigma_{V} = \frac{18.80}{9.25 - 2 \cdot (1.53)} \cdot \left(\frac{1}{2}\right) = 1.52 \text{ tsf} < 2.03 \text{ tsf} \quad \text{OK}
\]

The footing size is satisfactory for bearing.

**K. Check Sliding**  
**[10.6.3.3]**  
**[Table 3.11.5.3.1]**

The factored horizontal force is checked against the friction resistance between the footing and the soil. If adequate resistance is not provided by the footing, a shear key must be added.

\[
Q_R = \phi Q_n = \phi_T Q_T + \phi_{EP} Q_{EP}
\]

From LRFD Table 10.5.5-1, \( \phi_T = 0.80 \)
\[ Q_T = V \tan \delta \quad \text{(for cohesionless soils)} \]

[10.6.3.3] \[ Q_T = V \tan \delta = \tan \phi_f \quad \text{(for CIP footing)} \]

For Strength Ia:

\[ Q_T = 13.77 \tan 30^\circ = 7.95 \text{ kips} \]

\[ Q_{ep} = 0.0 \quad \text{(No shear key)} \]

\[ Q_R = 0.80 \times 7.95 + 0.0 = 6.36 \text{ kips} < 6.89 \text{ kips} \quad \text{NO GOOD} \]

A shear key must be added.

Similar to the retaining wall details in the Bridge Standard Plans, use a 1 foot by 1 foot shear key placed such that the back wall reinforcement will extend into the shear key. Consider only the passive resistance of soil in front of the shear key. Ignore the passive resistance of soil in front of the wall and toe. Refer to Figure 11.4.2.4.

*Passive Resistance Due to Shear Key*

*Figure 11.4.2.4*
For horizontal fill and a shear key with a vertical face,

\[ k_p = \tan^2 \left( 45 + \frac{\Phi_f}{2} \right) = \tan^2 \left( 45 + \frac{30}{2} \right) = 3.0 \]

Then:

\[ p_{ep1} = k_p w_s y_1 = 3.0 \cdot 0.120 \cdot 4.50 = 1.62 \text{ ksf} \]

\[ p_{ep2} = k_p w_s y_2 = 3.0 \cdot 0.120 \cdot 5.50 = 1.98 \text{ ksf} \]

\[ Q_{EP} = \left( \frac{p_{ep2} + p_{ep1}}{2} \right) (y_2 - y_1) \]

\[ = \left( \frac{1.98 + 1.62}{2} \right) \cdot (5.50 - 4.50) \]

\[ = 1.80 \text{ k} \]

Referring again to LRFD Table 10.5.5-1:

\[ \Phi_T = 1.00 \text{ (Soil on soil to be used only in area in front of shear key)} \]

\[ \Phi_{ep} = 0.50 \]

For Strength Ia with shear key added:

\[ Q_R = 1.00 \cdot \left( \frac{4.25}{9.25} \right) \cdot 7.95 + 0.80 \cdot \left( \frac{5.00}{9.25} \right) \cdot 7.95 + 0.50 \cdot 1.80 \]

\[ = 7.99 \text{ kips} > 6.89 \text{ kips} \quad \text{OK} \]

By inspection, sliding resistance is adequate for Extreme Event IIa.

**L. Design Footing for Shear [5.13.3.6]**

Design footings to have adequate shear capacity without transverse reinforcement.

**Determine \( d_v \)**

As a starting point, assume \#22 bars @ 12" (\( A_s = 0.60 \text{ in}^2/\text{ft} \)) for the top transverse bars in the heel and \#16 bars @ 12" (\( A_s = 0.31 \text{ in}^2/\text{ft} \)) for the bottom transverse bars in the toe. Cover is 3 inches for the top reinforcement and 5 inches for the bottom reinforcement.

Then for the heel:

\[ d_{sheel} = 18 - 3 - \frac{0.875}{2} = 14.56 \text{ in} \]

\[ a_{heel} = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} = \frac{0.60 \cdot 60}{0.85 \cdot 4 \cdot 12} = 0.88 \text{ in} \]
Check Heel for Shear

The critical shear section for the heel of the footing is located at the back face of the wall. The heel of the footing is assumed to carry its selfweight and the rectangular soil block above it. This neglects the benefit of any upward soil pressure below the footing.

\[ V_u = (\gamma_{EV} \cdot w_s \cdot h + \gamma_{DC} \cdot w_c \cdot c)(d - b - a) \]

\[ V_u = 1.35 \cdot 0.120 \cdot 13 + 1.25 \cdot 0.150 \cdot 1.50 \cdot (9.25 - 2.75 - 2.04) \]

\[ V_u = 10.65 \text{ kips} \]

Using \( \beta = 2.0 \) and assuming #22 bars in the top mat:

\[ \varphi \cdot V_c = \varphi \cdot 0.0316 \cdot \beta \cdot \sqrt{f'_c \cdot b_v \cdot d_v} \]

\[ \varphi \cdot V_c = 0.90 \cdot 0.0316 \cdot 2 \cdot \sqrt{4} \cdot 12 \cdot 14.12 \]

\[ = 19.28 \text{ k} > 10.65 \text{ k} \quad \text{OK} \]

Check Toe for Shear

The peak factored bearing stress is 1.52 tsf for the Extreme Event IIb load case. The critical section for the toe of the footing is at \( d_v \) from the front face of the wall. For a quick simplified check, try applying the peak bearing stress over the entire length of the toe (dimension \( b = 2' - 9'' \)).

\[ V_u = \sigma_v \cdot b = 1.52 \cdot 2.75 \cdot 2 = 8.36 \text{ k/ft} \]
By inspection the toe has adequate shear capacity. Each mat of reinforcement is checked to ensure that it has adequate capacity and that maximum and minimum reinforcement checks are satisfied.

The critical section for flexure in the footing is at the face of the wall.

**Top Transverse Reinforcement**

From the shear check of the heel, \( V_u = 10.65 \text{ kips} \).

Then:

\[
M_u = V_u \cdot (\text{moment arm}) = 10.65 \left( \frac{9.25 - 2.75 - 2.04}{2} \right) = 23.75 \text{ kip-ft}
\]

Set up the equation to solve for required steel area:

\[
M_u = \phi M_n = \phi \cdot A_s \cdot f_y \left( d_s - \frac{a}{2} \right)
\]

\[
= \phi \cdot A_s \cdot f_y \left( d_s - \frac{A_s \cdot f_y}{1.7 \cdot f_c \cdot b} \right)
\]

\[
M_u = 0.90 \cdot A_s \cdot 60 \cdot \left( d_s - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 12} \right) \cdot \left( \frac{1}{12} \right)
\]

\[
3.309 \cdot A_s^2 - 4.5 \cdot d_s \cdot A_s + M_u = 0
\]

For 3” clear cover and #22 bars, \( d_s = 14.56 \text{ in} \)

Substituting and solving for \( A_s \), we get:

Required \( A_s = 0.37 \text{ in}^2/\text{ft} \)

Try #19 bars @ 12”, \( A_s = 0.44 \text{ in}^2/\text{ft} \)

**Check Minimum Reinforcement**

Determine the cracking moment:

\[
f_r = 0.24 \cdot \sqrt{f_c} = 0.24 \cdot \sqrt{4} = 0.48 \text{ ksi}
\]

\[
I_g = \frac{1}{12} \cdot b \cdot c^3 = \frac{1}{12} \cdot 12 \cdot (18)^3 = 5832 \text{ in}^4
\]

\[
\gamma_t = \frac{1}{2} \cdot c = \frac{1}{2} \cdot 18 = 9 \text{ in}
\]
\[ M_{cr} = \frac{f_r \cdot I_g}{Y_t} = \frac{0.48 \cdot 5832}{9 \cdot (12)} = 25.9 \text{ kip-ft} \]

The capacity of the section must be \( \geq \) the smaller of:
\[ 1.2 M_{cr} = 31.1 \text{ kip-ft} \]

or \[ \frac{4}{3} M_u = \frac{4}{3} \cdot 23.75 = 31.7 \text{ kip-ft} \]

The capacity of the top mat of reinforcement is:
\[ M_r = \phi A_s f_y (d_s - a/2) \]
For #19 bars, \( d_s = 14.63 \text{ in} \)
\[ M_r = 0.9 \cdot (0.44) \cdot (60) \cdot \left[ 14.63 - \frac{0.44 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)} \right] \cdot \frac{1}{12} \]
\[ M_r = 28.3 \text{ kip-ft} < 31.1 \text{ kip-ft} \quad \text{NO GOOD} \]

Revise reinforcement to #22 bars @12", \( A_s = 0.60 \text{ in}^2/\text{ft} \)
\[ M_r = 38.1 \text{ kip-ft} > 31.1 \text{ kip-ft} \quad \text{OK} \]

[5.7.3.3.1] Check Maximum Reinforcement

The depth of the compression block is:
\[ c = \frac{A_s f_y}{0.85 f'_c \cdot \beta_1 \cdot b} = \frac{0.60 \cdot (60)}{0.85 \cdot (4) \cdot (0.85) \cdot (12)} = 1.04 \text{ in} \]
\[ \frac{c}{d_e} = \frac{c}{d_s} = \frac{1.04}{14.56} = 0.071 < 0.42 \quad \text{OK} \]

Use #22 bars @12" (\( A_s = 0.60 \text{ in}^2/\text{ft} \)) for top transverse reinforcement in the footing

**Bottom Transverse Reinforcement**

From shear check for the toe, \( V_u = 8.36 \text{ kips} \)
\[ M_u = V_u \cdot (\text{moment arm}) = 8.36 \cdot \left(\frac{2.75}{2}\right) = 11.50 \text{ kip-ft} \]

For 5" clear cover and #16 bars, \( d_s = 14.19 \text{ in} \)

Again use: \[ 3.309 \cdot A_s^2 - 4.5 \cdot d_s \cdot A_s + M_u = 0 \]
Substituting and solving for $A_s$, we get:

Required $A_s = 0.18 \text{ in}^2/\text{ft}$

Try #16 bars @12”, $A_s = 0.31 \text{ in}^2/\text{ft}$

**[5.7.3.3.1]**  
**Check Maximum Reinforcement**
Compute the compression block depth:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = \frac{0.31 \cdot (60)}{0.85 \cdot (4) \cdot (0.85) \cdot (12)} = 0.54 \text{ in}$$

$$\frac{c}{d_e} = \frac{c}{d_s} = \frac{0.54}{14.19} = 0.038 < 0.42 \quad \text{OK}$$

**[5.7.3.3.2]**  
**Check Minimum Reinforcement**

$$I_g = \frac{1}{12} \cdot b \cdot c^3 = \frac{1}{12} \cdot 12 \cdot (19.50)^3 = 7415 \text{ in}^4$$

$$y_t = \frac{1}{2} \cdot c = \frac{1}{2} \cdot 19.50 = 9.75 \text{ in}$$

$$M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.48 \cdot 7415}{9.75 \cdot (12)} = 30.4 \text{ kip-ft}$$

The capacity of the section must be ≥ the smaller of:

1.2 $M_{cr} = 1.2 \cdot 30.4 = 36.5 \text{ kip-ft}$

or $\frac{4}{3} \cdot M_U = \frac{4}{3} \cdot 11.50 = 15.3 \text{ kip-ft}$  \quad **GOVERNS**

Compute the capacity of the provided steel:

$$M_r = \phi A_s f_y (d_s - a/2)$$

$$M_r = 0.9 \cdot (0.31) \cdot (60) \left[ 14.19 - \frac{0.31 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)} \right] \cdot \left( \frac{1}{12} \right)$$

$$M_r = 19.48 \text{ ft-kips} > 15.3 \text{ kip-ft} \quad \text{OK}$$

*Use #16 bars @ 12” (A_s = 0.31 in^2/ft) for bottom transverse reinforcement in the footing.*
Longitudinal Reinforcement

[5.10.8]

Provide longitudinal reinforcement in the footing based on shrinkage and temperature requirements.

For footings, shrinkage reinforcement need not exceed 0.0015·A_g

\[ A_s = 0.0015 \cdot (19.50) \cdot (12) = 0.35 \text{ in}^2/\text{ft} \]

Total in each direction

Half is required in each face,

\[ \text{Min } A_s = \frac{1}{2} (0.35) = 0.18 \text{ in}^2/\text{ft} \]

Use #16 bars @ 12" (\(A_s = 0.31 \text{ in}^2/\text{ft}\)) for top and bottom longitudinal reinforcement in the footing.

N. Determine Loads For Wall Stem Design

The loads on the stem at the top of the footing can now be determined to arrive at the design forces for the wall.

Earth Pressure:

\[ P_{EH\text{top}} = 0 \text{ kips/ft} \]

\[ P_{EH\text{bottom}} = 0.033 \cdot (13.0) \cdot (1) = 0.429 \text{ kips/ft} \]

\[ P_{EH} = \frac{1}{2} (0.429) \cdot (13.0) = 2.79 \text{ kips} \]

\[ y_{EH} = \frac{13}{3} = 4.33 \text{ ft} \]

\[ M_{EH} = P_{EH} \cdot y_{LS} = 2.79 \cdot 4.33 = 12.08 \text{ kip-ft} \]

Live Load Surcharge:

\[ P_{LS} = (0.066) \cdot (13) = 0.86 \text{ kips} \]

\[ y_{LS} = \frac{13}{2} = 6.50 \text{ ft} \]

\[ M_{LS} = P_{LS} \cdot y_{LS} = 0.86 \cdot 6.50 = 5.59 \text{ kip-ft} \]

Using the Strength I load combination, the factored design forces for the wall stem are:

\[ H_u = 1.50 \cdot P_{EH} + 1.75 \cdot P_{LS} \]

\[ = 1.50 \cdot 2.79 + 1.75 \cdot 0.86 \]

\[ = 5.69 \text{ kips} \]

\[ M_u = 1.50 \cdot M_{EH} + 1.75 \cdot M_{LS} \]

\[ = 1.50 \cdot 12.08 + 1.75 \cdot 5.59 \]
= 27.90 kip-ft

The service design forces for the wall stem are:

\[ H_{\text{serv}} = 1.00 \cdot P_{\text{EH}} + 1.00 \cdot P_{\text{LS}} \]
\[ = 1.00 \cdot 2.79 + 1.00 \cdot 0.86 \]
\[ = 3.65 \text{ kips} \]

\[ M_{\text{serv}} = 1.00 \cdot M_{\text{EH}} + 1.00 \cdot M_{\text{LS}} \]
\[ = 1.00 \cdot 12.08 + 1.00 \cdot 5.59 \]
\[ = 17.67 \text{ kip-ft} \]

**O. Wall Stem Design – Investigate Shear**

Shear typically does not govern the design of retaining walls. If shear becomes an issue, the thickness of the stem should be increased such that transverse reinforcement is not required.

Ignoring the benefits of the shear key and axial compression, the shear capacity of the stem can be shown to be greater than that required.

\[ V_n = V_c + V_s + V_p \]

Recognizing that \( V_s \) and \( V_p \) are zero

\[ V_n = V_c \]

\[ V_c = 0.0316 \cdot \beta \cdot \sqrt{f'c} \cdot b_v \cdot d_v \]

Use \( d_v = 0.90 \cdot d_s \) and assume #25 bars

Then \( d_v = 0.90 \cdot (24.50 - 2.00 - 0.50) = 19.80 \text{ in} \)

Using \( \beta = 0.62 \) (smallest value in LRFD Table 5.8.3.4.2-2)

\[ V_c = 0.0316 \cdot 0.62 \cdot \sqrt{4} \cdot 12 \cdot 19.8 = 9.31 \text{ kips} \]

\[ \phi \cdot V_c = 0.90 \cdot 9.31 = 8.38 > 5.69 \text{ kips} \quad \text{OK} \]

**P. Wall Stem Design – Investigate Strength Limit State**

Determine the area of back-face flexural reinforcement necessary to satisfy the design moment:

\[ M_U = 27.90 \text{ kip-ft} \]
Again use the equation:

\[ 3.309 A_s^2 - 4.5 d_s A_s + M_u = 0 \]

For 2 inch clear cover and #25 bars, \( d_s = 22.00 \) in

Substitute and solve for \( A_s \):

Required \( A_s = 0.28 \) in\(^2/\)ft

Try #16 bars @12” (\( A_s = 0.31 \) in\(^2/\)ft.)

**Q. Wall Design – Investigate Service Limit State [5.7.3.4]**

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. Design for a \( z \) value of 170 kip/in. The service load bending moment is 17.67 kip-ft.

For #16 bars @ 12”, \( A_s = 0.31 \) in\(^2/\)ft:

\[
\begin{align*}
d_s &= 24.50 - 2.00 - \frac{0.625}{2} = 22.19 \text{ in} \\
d_c &= 2.00 + \frac{0.625}{2} = 2.31 \text{ in} \\
A &= \frac{2 \cdot (d_c) \cdot b}{N} = \frac{2 \cdot (2.31) \cdot 12}{1} = 55.44 \text{ in}^2
\end{align*}
\]

The allowable stress, \( f_{sa} \) is:

\[
f_{sa} = \frac{z}{\sqrt[3]{d_c \cdot A}} = \frac{170}{\sqrt[3]{2.31 \cdot 55.44}} = 33.7 \text{ ksi GOVERSNS}
\]

or \( f_{sa} = 0.6 \cdot f_y = 0.6 \cdot 60 = 36.0 \text{ ksi} \)

[5.4.2.4] The transformed area of reinforcement is:

\[ n \cdot A_s = 8 \cdot (0.31) = 2.48 \text{ in}^2 \]
Determine location of the neutral axis:

\[
\frac{1}{2} \cdot b x^2 = n \cdot A_s (d_s - x)
\]

\[
\frac{1}{2} \cdot (12) \cdot x^2 = 2.48 (22.19 - x) \quad \text{solving, } x = 2.83 \text{ inches}
\]

Then:

\[
j \cdot d_s = \frac{d_s - x}{3} = 22.19 - \frac{2.83}{3} = 21.25 \text{ in}
\]

Actual \( f_s = \frac{M}{A_s \cdot j \cdot d_s} = \frac{17.67 \cdot 12}{0.31 \cdot (21.25)} \]

\[
= 32.2 \text{ ksi} < 33.7 \text{ ksi} \quad \text{OK}
\]

**R. Check Reinforcement Limits**

**Check Minimum Reinforcement**

Minimum reinforcement is required to ensure that ductile behavior occurs at ultimate load. The flexural reinforcement needs to be able to carry the lesser of 1.20 times the cracking moment or 1.33 times the original design moment.

\[
f_r = 0.24 \cdot \sqrt{f_c} = 0.24 \cdot \sqrt{4} = 0.48 \text{ ksi}
\]

\[
I_g = \frac{1}{12} b \cdot a^3 = \frac{1}{12} \cdot 12 \cdot (24.5)^3 = 14,706 \text{ in}^4
\]
\[ y_t = \frac{1}{2} \cdot a = \frac{1}{2} \cdot 24.50 = 12.25 \text{ in} \]

\[ M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.48 \cdot 14,706}{12.25 \cdot (12)} = 48.0 \text{ kip-ft} \]

The capacity of the section must be \( \geq \) the smaller of:

\[ 1.2 \ M_{cr} = 57.6 \text{ kip-ft} \]

or \[ \frac{4}{3} \cdot M_u = \frac{4}{3} \cdot 27.90 = 37.2 \text{ kip-ft} \]\text{ GOVERSNS}

\[ M_r = \phi \cdot A_s \cdot f_y \left( d_s - \frac{a}{2} \right) / 12 \]

\[ M_r = 0.9 \ (0.31) \cdot (60) \left[ 22.19 - \frac{0.31 \cdot (60)}{2 \cdot (0.85) \cdot (4) \cdot (12)} \right] \cdot \frac{1}{12} \]

\[ M_r = 30.6 \text{ kip-ft} < 37.2 \text{ kip-ft} \]\text{ NO GOOD}

Revise reinforcement to #19 bars @ 12”, \( A_s = 0.44 \text{ in}^2/\text{ft} \)
\[ d_s = 22.13 \text{ in} \]
\[ M_r = 43.2 \text{ kip-ft} > 37.2 \text{ kip-ft} \]\text{ OK}

Check Maximum Reinforcement
To reduce the likelihood of a brittle failure mode a limit is placed on the maximum flexural reinforcement allowed in the cross section.

\[ c = \frac{A_s \cdot f_y}{0.85 \cdot f_c' \cdot \beta_1 \cdot b} = \frac{0.44 \cdot (60)}{0.85 \cdot (4) \cdot (0.85) \cdot (12)} = 0.76 \text{ in} \]

\[ d_e = d_s = 22.13 \text{ inches} \]

\[ \frac{c}{d_e} = \frac{0.76}{22.13} = 0.034 < 0.42 \]\text{ OK}

Use #19 bars @ 12” (\( A_s = 0.44 \text{ in}^2/\text{ft} \)) for stem wall back face vertical bars.

\[ S. \ Design \ Stem \ Wall \ Shrinkage \ and \ Temperature \ Reinforcement \ [5.10.8] \]

To ensure good performance, a minimum amount of reinforcement needs to be placed near each face of concrete elements. This reinforcement limits the size of cracks associated with concrete shrinkage and temperature changes.
Min. Total Temp. \( A_s \geq 0.11 \left[ \frac{A_g}{f_y} \right] = 0.11 \left[ \frac{24.5 \cdot (12)}{60} \right] = 0.54 \text{ in}^2/\text{ft} 

For walls, temperature \( A_s \) need not exceed 0.0015 \( A_g \):

Min. Total Temp. \( A_s \geq 0.0015 \cdot (24.5) \cdot (12) = 0.44 \text{ in}^2/\text{ft} \quad \text{GOVERNS} 

This is the total area in each direction. It is distributed to both faces of the element.

Based on half placed in each face, the required steel for each face is:

Required Temp. \( A_s = \frac{1}{2} (0.44) = 0.22 \text{ in}^2/\text{ft} \) each direction

Use \#16 @ 12\" (\( A_s = 0.31 \text{ in}^2/\text{ft} \)) for stem wall front face bars and back face horizontal bars.

**T. Summary**

The wall section shown in Figure 11.4.2.6 summarizes the design of the retaining wall. Note that the spacing of the longitudinal footing bars is revised slightly from previous calculations for detailing purposes.
Figure 11.4.2.6
This example illustrates the design of a reinforced concrete three-column pier. The pier supports a prestressed beam superstructure. The design of the beams is presented in Section 5.7.2. The bearings are designed in Section 14.8.1. The superstructure has two equal spans of 130 feet and is part of a grade-separation structure. The superstructure is considered translationally fixed at the pier. An end view of the pier is presented in Figure 11.4.3.1. Two sets of bearings rest on the pier cap, one set for the beams of each span. To simplify design, only one reaction is used per beam line, acting at the centerline of pier.
The pier cap is supported by three columns. The columns are supported by separate pile foundations. An elevation view of the pier is presented in Figure 11.4.3.2.

3-Column Pier - Elevation
Figure 11.4.3.2
Pier design is accomplished with a top down approach. The design parameters and loads are determined first - followed by the pier cap, column, and footing designs.

The following terms are used to describe the orientation of the structural components and loads. The terms “longitudinal” and “transverse” are used to describe global orientation relative to the superstructure and roadway. The terms “parallel” and “perpendicular” are used to define the orientations relative to the pier. The parallel dimension is the “long” direction of the structural component and the perpendicular dimension is 90° to the parallel dimension and is in the direction of the “short” side. The distinction becomes clear in describing the load path for lateral forces applied to bridges with substructures skewed to the superstructure. Forces parallel and perpendicular to the pier arise from combining the component forces applied transversely and longitudinally to the superstructure. The pier for this example is not skewed, consequently transverse forces are equivalent to parallel pier forces. However, to ensure the clarity of future designs, the parallel and perpendicular nomenclature will be used.

### A. Material and Design Parameters

**Pier Cap**

The cap must have sufficient length to support all of the beam lines and their bearings. It also must have sufficient width to support two lines of bearings and provide adequate edge distances for the bearings. Pedestals are constructed on the pier cap to accommodate the different heights at which the prestressed beams are supported due to the cross slope of the deck. When beginning a design, first determine the required width and then try a cap depth equal to 1.4 to 1.5 times the width.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Label</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of Pier Cap</td>
<td>b&lt;sub&gt;cap&lt;/sub&gt;</td>
<td>40 in</td>
</tr>
<tr>
<td>Length of Pier Cap</td>
<td>L&lt;sub&gt;cap&lt;/sub&gt;</td>
<td>51 ft = 612 in</td>
</tr>
<tr>
<td>Depth of Pier Cap at Center</td>
<td>d&lt;sub&gt;mid&lt;/sub&gt;</td>
<td>56 in</td>
</tr>
<tr>
<td>Depth of Pier Cap at Ends</td>
<td>d&lt;sub&gt;end&lt;/sub&gt;</td>
<td>36 in</td>
</tr>
</tbody>
</table>

**Columns**

In order to avoid interference between the column vertical bars and pier cap reinforcement, choose columns with a diameter slightly smaller than the width of the pier cap. Columns should also be proportioned relative to the depth of the superstructure. For 72” prestressed beams a column diameter of at least 36 inches should be used. (See Section 11.1.2.)
Table 11.4.3.2 – Column Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Label</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column Diameter</td>
<td>(d_{col})</td>
<td>36 in</td>
</tr>
<tr>
<td>Number of Columns</td>
<td>(N_{col})</td>
<td>3</td>
</tr>
<tr>
<td>Column Cross-Sectional Area</td>
<td>(A_g)</td>
<td>(\frac{\pi \cdot 36^2}{4} = 1018 \text{ in}^2)</td>
</tr>
<tr>
<td>Column Moment of Inertia</td>
<td>(I_g)</td>
<td>(\frac{\pi \cdot 36^4}{64} = 82,450 \text{ in}^4)</td>
</tr>
</tbody>
</table>

**Footing and Piles**

A rectangular footing with the following properties will be tried initially:

Table 11.4.3.3 – Foundation Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Label</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile Type</td>
<td>-</td>
<td>Cast-In-Place</td>
</tr>
<tr>
<td>Pile Diameter</td>
<td>(d_{pile})</td>
<td>12 in</td>
</tr>
<tr>
<td>Depth of Footing</td>
<td>(d_{foot})</td>
<td>3.75 ft</td>
</tr>
<tr>
<td>Width of Footing Parallel to Pier</td>
<td>(b_{foot})</td>
<td>9.0 ft</td>
</tr>
<tr>
<td>Length of Footing Perpendicular to Pier</td>
<td>(L_{foot})</td>
<td>12.0 ft</td>
</tr>
</tbody>
</table>

The Bridge Construction Unit’s Foundation Recommendations must be referenced at the beginning of final design. The recommendations identify the pile design capacity \(Q_n\) and resistance factor \(\phi\) to be used:

Nominal Capacity \(Q_n = 180 \text{ tons/pile}\)
Resistance Factor \(\phi = 0.45\)
Bearing Resistance \(Q_r = \phi \cdot Q_n = 0.45 \cdot 180 = 81 \text{ tons/pile} = 162 \text{ kips/pile}\)

**Location of Columns**

The outside columns should be positioned to minimize dead load moments in the columns and also balance the negative moments in the pier cap over the columns. A rule of thumb is to use an overhang dimension (measured from edge of outside column to centerline of exterior beam) equal to \(\frac{1}{5}\) of the column spacing. After trying several layouts, outside columns located 18.75 feet from the center of the bridge were found to minimize design forces.
The following material weights and strengths are used in this example:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Label</th>
<th>Value</th>
</tr>
</thead>
</table>
| Unit Weight of Concrete            | $\gamma_c$ | 0.145 kcf (strength)  
                               |       | 0.150 kcf (loads)                          |
| Concrete Compressive Strength      | $f'_c$ | 4 ksi                                       |
| Modulus of Elasticity, Concrete    | $E_c$ | $33,000 \cdot (0.145)^{1.5} \cdot \sqrt{4}$  
                               |       | = 3644 ksi                                  |
| Yield Strength of Reinforcement    | $f_y$ | 60 ksi                                      |
| Modulus of Elasticity, Reinforcement | $E_s$ | 29,000 ksi                                  |
| Modular Ratio                      | $n$   | 8                                           |
| Soil Unit Weight                   | $\gamma_{soil}$ | 0.120 kcf                                |

**B. Determine Design Loads**

The loads applied to the three-column pier include dead load, live load, braking force, wind on structure, wind on live load, and uniform temperature change. The pier is assumed to be protected by one of the means identified in LRFD Article 3.6.5.1. Vehicular collision forces will not be considered.

**Application of Loads to the Structural Model**

Aside from wind on substructure and internal temperature change forces, the loads applied to the pier are transferred from the superstructure to the pier cap via the bearings. Figure 11.4.3.3 illustrates the load components that are transferred from the bearings to the pier cap. At each girder location three load components are possible, a parallel force, a perpendicular force, and a vertical force. In the following load tables, vertical force components are identified as $V_1$ to $V_6$. Parallel forces have labels of $L_{Par1}$ to $L_{Par6}$, and perpendicular forces are identified as $L_{Perp1}$ to $L_{Perp6}$.

For several loads applied to the pier, the concrete deck was assumed to be a rigid diaphragm. A rigid deck assumption combined with the presence of diaphragms at the pier permits one to assume that the parallel and perpendicular wind loads can be evenly distributed among the bearings. Varying vertical reactions resist lateral and vertical loads that produce an overturning moment.
The superstructure dead loads applied to the pier consist of the following: the design shear in the prestressed beam at the centerline of bearing, the beam ends (portion of the beams beyond centerline of bearing), the portion of deck, stool, barrier, and future wearing course between centerline of bearings, the cross-frames at the pier, two sets of bearings per beam line, and the pedestals. The additional dead load is approximately 5 kips for the fascia beams and 6 kips for the interior beams.

**Table 11.4.3.5 - Superstructure Dead Loads (kips)**

<table>
<thead>
<tr>
<th>Load/Location</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
<th>V5</th>
<th>V6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstructure/Bearings/Pedestals</td>
<td>284</td>
<td>298</td>
<td>298</td>
<td>298</td>
<td>298</td>
<td>284</td>
</tr>
</tbody>
</table>
Live Load
First, the maximum reaction at the pier due to a single lane of HL-93 live load must be determined. After comparing results from several configurations, the double truck with lane load shown in Figure 11.4.3.4 was found to produce the largest reaction.

Table 11.4.3.6 lists the live load reactions at the pier for different numbers of lanes loaded. It also includes the maximum reaction for fatigue, which occurs when the center axle of the fatigue truck is directly over the pier.

Table 11.4.3.6 – Live Load Reactions on Pier (per lane)

<table>
<thead>
<tr>
<th>Loading</th>
<th>Truck Load Reaction with Dynamic Load Allowance (kips)</th>
<th>Lane Load Reaction (kips)</th>
<th>Product of Multiple Presence Factor and Double Truck Load Factors</th>
<th>Total Reaction R (kips)</th>
<th>Uniform Load w = R/10’ (kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Lane</td>
<td>134.1 - Double Truck</td>
<td>83.2</td>
<td>1.20·0.90=1.08</td>
<td>234.7</td>
<td>23.5</td>
</tr>
<tr>
<td>2 Lanes</td>
<td>134.1 – Double Truck</td>
<td>83.2</td>
<td>1.00·0.90=0.90</td>
<td>195.6</td>
<td>19.6</td>
</tr>
<tr>
<td>3 Lanes</td>
<td>134.1 - Double Truck</td>
<td>83.2</td>
<td>0.85·0.90=0.765</td>
<td>166.2</td>
<td>16.6</td>
</tr>
<tr>
<td>4 Lanes</td>
<td>134.1 - Double Truck</td>
<td>83.2</td>
<td>0.65·0.90=0.585</td>
<td>127.1</td>
<td>12.7</td>
</tr>
<tr>
<td>Fatigue</td>
<td>73.3 – Fatigue Truck</td>
<td>0.0</td>
<td>1.00</td>
<td>73.3</td>
<td>7.3</td>
</tr>
</tbody>
</table>
The next step is to determine the live load cases that will produce the maximum force effects in the cap, columns, and foundation of the pier. This is done by positioning the single lane reactions in lanes across the transverse bridge cross-section to get the desired effect.

For instance, to obtain the maximum positive moment in the pier cap, place one or two live load lane reactions on the deck such that the beams located between the columns receive the maximum load. Figure 11.4.3.5 illustrates the live load cases used in the example. Table 11.4.3.7 contains beam reactions for each of the load cases. Load distribution for determination of values in the table is based on assuming simple supports at each beam.

For example, for Live Load Case 2:

\[ w = 23.5 \text{ kips/ft} \]

\[ V_1 = V_6 = 0 \]

\[ V_2 = V_5 = 23.5 \cdot \left( \frac{9 - 8.50}{2} \right)^2 \cdot \left( \frac{1}{9} \right) = 0.3 \text{ kips} \]

\[ V_3 = V_4 = 23.5 \cdot \frac{1}{2} + 23.5 \cdot 0.5 \cdot 8.75 \cdot \frac{1}{9} = 117.2 \text{ kips} \]
Figure 11.4.3.5
Table 11.4.3.7 - Superstructure Live Load Beam Reactions (kips)
(includes dynamic load allowance)

<table>
<thead>
<tr>
<th>Live Load Case</th>
<th>Location</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
<th>V5</th>
<th>V6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>One Lane Positive Cap Moment</td>
<td>1.0</td>
<td>125.4</td>
<td>108.6</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>One Lane Over Center Column</td>
<td>0.0</td>
<td>0.3</td>
<td>117.2</td>
<td>117.2</td>
<td>0.3</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>One Lane At Gutter Line</td>
<td>143.1</td>
<td>94.3</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>4</td>
<td>Two Lanes Positive Cap Moment</td>
<td>37.6</td>
<td>165.8</td>
<td>160.0</td>
<td>28.5</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>5</td>
<td>Two Lanes Over Center Column</td>
<td>0.0</td>
<td>32.9</td>
<td>163.1</td>
<td>163.1</td>
<td>32.9</td>
<td>0.0</td>
</tr>
<tr>
<td>6</td>
<td>Three Lanes Over Center Column</td>
<td>5.8</td>
<td>108.6</td>
<td>134.6</td>
<td>134.6</td>
<td>108.6</td>
<td>5.8</td>
</tr>
<tr>
<td>7</td>
<td>Four Lanes</td>
<td>51.0</td>
<td>95.8</td>
<td>107.2</td>
<td>107.2</td>
<td>95.8</td>
<td>51.0</td>
</tr>
<tr>
<td>8</td>
<td>Fatigue-One Lane Positive Cap Moment</td>
<td>0.3</td>
<td>39.0</td>
<td>33.7</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>9</td>
<td>Fatigue–One Lane Over Center Column</td>
<td>0.0</td>
<td>0.1</td>
<td>36.4</td>
<td>36.4</td>
<td>0.1</td>
<td>0.0</td>
</tr>
<tr>
<td>10</td>
<td>Fatigue – One lane At Gutter Line</td>
<td>44.6</td>
<td>29.3</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

**Braking Force**

For this example, braking forces due to four lanes of traffic are assumed to transmit a longitudinal (perpendicular to the pier) force that is evenly distributed to the three columns.

[3.6.4]

Begin by determining if a truck by itself or if truck plus lane loading governs the braking force.

**Truck alone:**

\[0.25 \cdot (8 + 32 + 32) = 18.0 \text{ kips}\]

**Truck plus lane:**

\[0.05 \cdot [8 + 32 + 32 + (2 \cdot 130 \cdot 0.64)] = 11.9 \text{ kips}\]

Then the design force is:

\[\text{BR} = 18.0 \cdot (# \text{ of lanes}) \cdot (\text{multiple presence factor})\]

\[= 18.0 \cdot 4 \cdot 0.65 = 46.8 \text{ kips}\]

Although the lateral braking force is to be applied 6 feet above the top of deck, it gets transferred to the pier through the bearings. In order to satisfy statics and make the two load systems equivalent, transfer of the lateral force down to the bearing level requires the addition of a moment couple equal to:

\[L_{BR} \cdot [6 \text{ ft} + (\text{distance from top of deck to bearings})]\]

Figure 11.4.3.6 illustrates this. The moment couple consists of vertical forces at the abutments. Because the distance from abutment to
abutment is very large relative to the transfer height, the vertical forces are negligible and will be ignored. Therefore, we can conclude that for pier analysis, the braking force can be applied at the top of the pier.

Equivalent Load Systems

**Figure 11.4.3.6**

Height of load application $y_{BR}$ above the top of footing is:

$$y_{BR} = 26.00 - 3.75 = 22.25 \text{ ft}$$

The moment at the base of the columns is:

$$M_{perpBR} = 46.8 \cdot 22.25 \cdot \left(\frac{1}{3}\right) = 341.1 \text{ kip-ft/column}$$

Wind on Superstructure

For the wind load on the superstructure, the deck functions as a horizontal 2-span continuous beam with wind pressure acting on the exposed edge area of the superstructure. The reaction at the fixed end for a propped cantilever beam is $5/8$ of the uniformly applied load. Then for a 2-span continuous beam, $5/8$ of wind from both spans is carried by the pier and the tributary area for superstructure wind is:

$$A_{wsup} = 2 \cdot \left(\frac{5}{8}\right) \cdot 130 \cdot 9.75 = 1584 \text{ ft}^2$$

The wind on superstructure load $W_{S\text{sup}}$ is:

$$W_{S\text{sup}} = P_B \cdot A_{wsup}$$

where $P_B$ = base wind pressure from LRFD Table 3.8.1.2.2-1 for various attack angles.

For example, for a wind attack angle skewed 30 degrees to the superstructure:

$$P_{B\text{transv}} = P_{B\text{par}} = 0.041 \text{ ksf}$$

$$P_{B\text{long}} = P_{B\text{perp}} = 0.012 \text{ ksf}$$
Similar to the braking force, the longitudinal wind component on the superstructure can be applied at the top of the pier for analysis. The height of application \( y_{\text{perp}} \) above the top of footing is:

\[
y_{\text{perp}} = 26.00 - 3.75 = 22.25 \text{ ft}
\]

Then, for a wind attack angle skewed 30 degrees to the superstructure, the moment at the base of the columns is:

\[
M_{w_{\text{supperp}}} = 19.0 \cdot 22.25 \left( \frac{1}{3} \right) = 140.9 \text{ kip} - \text{ft/column}
\]

For the analysis model, the transverse wind component will be applied at the centroid of the pier cap. Transfer of the transverse wind component from the centroid of the exposed superstructure area to the centroid of the pier cap requires the addition of vertical loads at the bearings equivalent to the reduction in moment \( M_{\text{red}} \). For a wind attack angle skewed 30 degrees:

\[
m_{\text{red}} = W_{s_{\text{suppar}}} \cdot (\text{dist. from superstr. centroid to pier cap centroid})
\]

\[
= 64.9 \left( \frac{9.75}{2} + \frac{4.67}{2} \right) = 467.9 \text{ kip} - \text{ft}
\]

The additional vertical loads are calculated assuming the moment is applied at the center of the bridge and the deck is rigid. The “I” of the beams is determined and vertical loads “\( V \)” are based on the formula:

\[
V = \frac{M_{\text{red}} x_{\text{beam}}}{I_{\text{beams}}}
\]

where \( x_{\text{beam}} \) = distance from center of bridge to centerline of beam

and \( I_{\text{beams}} = \sum x_{\text{beam}}^2 \)

Then for Beam 1 (left fascia beam with vertical load \( V_1 \)) and a wind attack angle skewed 30 degrees:

\[
x_{\text{beam1}} = 22.5 \text{ ft}
\]

\[
I_{\text{beams}} = \sum x^2 = (22.5)^2 + (13.5)^2 + (4.5)^2 + (-4.5)^2 + (-13.5)^2 + (-22.5)^2
\]

\[
= 1417.5 \text{ ft}^2
\]
The wind on superstructure loads applied to the pier are summarized in Table 11.4.3.8.

### Table 11.4.3.8 – Wind on Superstructure Loads

<table>
<thead>
<tr>
<th>Wind Attack Angle</th>
<th>( L_{\text{par}1} V_1 ) (kips)</th>
<th>( L_{\text{par}2} V_2 ) (kips)</th>
<th>( L_{\text{par}3} V_3 ) (kips)</th>
<th>( L_{\text{par}4} V_4 ) (kips)</th>
<th>( L_{\text{par}5} V_5 ) (kips)</th>
<th>( L_{\text{par}6} V_6 ) (kips)</th>
<th>( M_{\text{wupperp}} ) (kip-ft/col)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Degree Skew</td>
<td>13.2</td>
<td>9.1</td>
<td>13.2</td>
<td>13.2</td>
<td>13.2</td>
<td>13.2</td>
<td>0.0</td>
</tr>
<tr>
<td>15 Degree Skew</td>
<td>11.6</td>
<td>8.0</td>
<td>11.6</td>
<td>11.6</td>
<td>11.6</td>
<td>11.6</td>
<td>70.5</td>
</tr>
<tr>
<td>30 Degree Skew</td>
<td>10.8</td>
<td>7.4</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>10.8</td>
<td>140.9</td>
</tr>
<tr>
<td>45 Degree Skew</td>
<td>8.7</td>
<td>6.0</td>
<td>8.7</td>
<td>8.7</td>
<td>8.7</td>
<td>8.7</td>
<td>187.6</td>
</tr>
<tr>
<td>60 Degree Skew</td>
<td>4.5</td>
<td>3.1</td>
<td>4.5</td>
<td>4.5</td>
<td>4.5</td>
<td>4.5</td>
<td>223.2</td>
</tr>
</tbody>
</table>

[3.8.2]

An additional wind on superstructure load is considered for the case where there is no live load and the wind is oriented at 0 degrees. The load represents uplift on the bottom of the deck and is called the vertical wind on superstructure load. The deck was assumed hinged over the pier for this load case. A tributary length of 130 feet (2·65 feet) was used with the deck width of 51.33 feet. A 0.020 ksf pressure produces a vertical force of:

**Vertical force:**

\[ WS_v = 51.33 \cdot -0.020 \cdot 130 = -133.5 \text{ kips} \]

**Eccentricity of Vertical force:**

\[ e_{wsv} = \frac{51.33}{4} = 12.83 \text{ ft} \]

**Overturning Moment:**

\[ M_{wsv} = 133.5 \cdot 12.83 = 1713 \text{ k-ft} \]

The vertical force applied to the pier at each bearing location can be calculated using the formula:

\[ V = \frac{WS_v + M_{wsv} \times beam}{N \times I_{beams}} \]

where \( N \) = number of beams
For example, at the bearing location for Beam 1 (left fascia beam with vertical load $V_1$):

$$V_1 = \frac{-133.5}{6} + \frac{1713 \times (22.5)}{1417.5} = 4.9 \text{ kips}$$

Table 11.4.3.9 summarizes the vertical wind on superstructure loads.

**Table 11.4.3.9 – Vertical Wind on Superstructure Loads (kips)**

<table>
<thead>
<tr>
<th>Wind Attack Angle</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
<th>V5</th>
<th>V6</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Degree Skew</td>
<td>4.9</td>
<td>-5.9</td>
<td>-16.8</td>
<td>-27.7</td>
<td>-38.6</td>
<td>-49.4</td>
</tr>
</tbody>
</table>

**[3.8.1.2.3]**

**Wind on Substructure**

A wind load with a base wind pressure $P_B$ of 0.040 ksf, resolved into components for wind attack angles that are skewed, is applied directly to the pier. See Table 11.4.3.10. This wind load was applied as line loads to the pier cap and column members in the structural analysis model. Assuming 1 foot of cover over the tops of the footings, the projected area of the perpendicular face is 65.3 ft$^2$. The parallel face has an area of 229.4 ft$^2$ for the pier cap and 149.2 ft$^2$ for the columns.

**Table 11.4.3.10 – Wind on Substructure (ksf)**

<table>
<thead>
<tr>
<th>Wind Attack Angle</th>
<th>Pressure on Perpendicular Face</th>
<th>Pressure on Parallel Face</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Degree Skew</td>
<td>0.040</td>
<td>0.000</td>
</tr>
<tr>
<td>15 Degree Skew</td>
<td>0.039</td>
<td>0.010</td>
</tr>
<tr>
<td>30 Degree Skew</td>
<td>0.035</td>
<td>0.020</td>
</tr>
<tr>
<td>45 Degree Skew</td>
<td>0.028</td>
<td>0.028</td>
</tr>
<tr>
<td>60 Degree Skew</td>
<td>0.020</td>
<td>0.035</td>
</tr>
</tbody>
</table>

**[3.8.1.3]**

**Wind on Live Load**

The wind on live load was assumed to have the same tributary length as the wind on superstructure load. It is applied at 6 feet above the top of the deck.

Tributary length for wind on live load:

$$L_{trib} = 2 \times 130 \times \frac{5}{8} = 162.5 \text{ ft}$$

The wind on live load $WL$ is:

$$WL = P_B \cdot L_{trib}$$

where $P_B =$ base wind pressure from LRFD Table 3.8.1.3-1 for various wind attack angles.
For example, for a wind attack angle skewed 30 degrees to the superstructure:

\[ P_{B\text{transv}} = P_{\text{spar}} = 0.082 \text{ klf} \]
\[ P_{B\text{long}} = P_{B\text{perp}} = 0.024 \text{ klf} \]
\[ WL_{\text{par}} = 0.082 \cdot 162.5 = 13.3 \text{ kips} \]

Then the lateral load \( L \) applied to the pier cap at each beam location is:

\[ L_{\text{par}1} = L_{\text{par}2} = L_{\text{par}3} = L_{\text{par}4} = L_{\text{par}5} = L_{\text{par}6} = \frac{13.3}{6} = 2.2 \text{ kips} \]
\[ WL_{\text{perp}} = 0.024 \cdot 162.5 = 3.9 \text{ kips} \]

Similar to the wind on superstructure load, the longitudinal wind on live load component can be applied at the top of the pier for analysis. The height of application \( y_{\text{perp}} \) above the top of footing is:

\[ y_{\text{perp}} = 26.00 - 3.75 = 22.25 \text{ ft} \]

Then for a wind attack angle skewed 30 degrees to the superstructure, the moment at the base of the columns is:

\[ M_{WL\text{perp}} = 3.9 \cdot 22.25 \cdot \left( \frac{1}{3} \right) = 28.9 \text{ kip} \cdot \text{ft/column} \]

Again, similar to the wind on superstructure load, the transverse wind on live load component will be applied at the centroid of the pier cap. This will require the addition of vertical loads at the bearings equivalent to the reduction in moment. For a wind attack angle skewed 30 degrees:

\[ M_{\text{red}} = WL_{\text{par}} \cdot (6 \text{ ft} + \text{dist. from top of deck to pier cap centroid}) \]
\[ = 13.3 \cdot \left( 6.00 + (9.75 - 2.67) + \frac{4.67}{2} \right) = 205.0 \text{ kip} \cdot \text{ft} \]

Then for Beam 1 (left fascia beam with vertical load \( V_1 \)) and a wind attack angle skewed 30 degrees:

\[ V_1 = \frac{M_{\text{red}}x_{\text{beam}}}{I_{\text{beams}}} = \frac{205.0 (22.5)}{1417.5} = 3.3 \text{ kips} \]

The wind on live load values are summarized in Table 11.4.3.11.
Table 11.4.3.11 – Wind on Live Load

<table>
<thead>
<tr>
<th>Wind Attack Angle</th>
<th>L_{par1} V_1 (kips)</th>
<th>L_{par2} V_2 (kips)</th>
<th>L_{par3} V_3 (kips)</th>
<th>L_{par4} V_4 (kips)</th>
<th>L_{par5} V_5 (kips)</th>
<th>L_{par6} V_6 (kips)</th>
<th>M_WLperp (kip-ft/col)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Degree Skew</td>
<td>2.7</td>
<td>2.7</td>
<td>2.7</td>
<td>2.7</td>
<td>2.7</td>
<td>2.7</td>
<td>0.0</td>
</tr>
<tr>
<td>15 Degree Skew</td>
<td>2.4</td>
<td>2.4</td>
<td>2.4</td>
<td>2.4</td>
<td>2.4</td>
<td>2.4</td>
<td>14.8</td>
</tr>
<tr>
<td>30 Degree Skew</td>
<td>2.2</td>
<td>2.2</td>
<td>2.2</td>
<td>2.2</td>
<td>2.2</td>
<td>2.2</td>
<td>28.9</td>
</tr>
<tr>
<td>45 Degree Skew</td>
<td>1.8</td>
<td>1.8</td>
<td>1.8</td>
<td>1.8</td>
<td>1.8</td>
<td>1.8</td>
<td>38.6</td>
</tr>
<tr>
<td>60 Degree Skew</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>46.0</td>
</tr>
</tbody>
</table>

C. Structural Analysis

To determine the design forces in different portions of the pier a structural analysis was performed with a matrix analysis program. Gross section properties were used for all members. Fixed supports were provided at the top of each footing.

D. Design of the Pier Cap

1. Design Loads

The pier cap is designed for dead and live loads. Wind and truck braking loads are assumed not to contribute to maximum vertical load effects for the design of the pier cap. Four load combinations are examined (Strength I, Strength IV, Service I, and Fatigue). Strength I and IV are used to determine basic flexural and shear demands:

\[ U_1 = 1.25 \cdot DC + 1.75 \cdot (L + IM) \]
\[ U_2 = 1.5 \cdot DC \]

Service I is used to check the distribution of flexural reinforcement (crack control):

\[ S_1 = 1.00 \cdot DC + 1.00 \cdot (L + IM) \]

The Fatigue limit state is used to ensure that adequate fatigue resistance is provided.

\[ F = 0.75 \cdot (L + IM) \]

The pier cap design forces are listed in Tables 11.4.3.12 and 11.4.3.13.
For simplicity, negative bending moments are given at the column centerline. Another reasonable approach would be to use the average of the moments at the column centerline and the column face for the design negative moment.

Again for simplicity, pier cap shears at the columns are given at the column centerline. For pier configurations where beam reactions are located over a column, the design shear should be taken at the column face.

### Table 11.4.3.12 – Pier Cap Design Moments

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Positive Bending Moment (located at CL Beam 2) (kip-ft)</th>
<th>Negative Bending Moment (located at CL Column 1) (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I</td>
<td>1563 (LL Case 4)</td>
<td>2337 (LL Case 3)</td>
</tr>
<tr>
<td>Strength IV</td>
<td>689</td>
<td>1691</td>
</tr>
<tr>
<td>Service I</td>
<td>1025 (LL Case 4)</td>
<td>1655 (LL Case 3)</td>
</tr>
<tr>
<td>Fatigue (max)</td>
<td>113 (LL Case 8)</td>
<td>125 (LL Case 10)</td>
</tr>
<tr>
<td>Fatigue (min)</td>
<td>-4 (LL Case 10)</td>
<td>0</td>
</tr>
<tr>
<td>Permanent Loads</td>
<td>459</td>
<td>1118</td>
</tr>
</tbody>
</table>

### Table 11.4.3.13 – Pier Cap Design Shears (kips)

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Location Along Pier Cap</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CL Beam 1</td>
</tr>
<tr>
<td>Strength I</td>
<td>614 (LL Case 3)</td>
</tr>
<tr>
<td>Strength IV</td>
<td>437</td>
</tr>
<tr>
<td>Permanent Loads</td>
<td>291</td>
</tr>
</tbody>
</table>

2. Design Positive Moment Reinforcement

The flexural design of the cap is accomplished with five checks: flexural strength, crack control, fatigue, maximum reinforcement and minimum reinforcement. An appropriate level of reinforcement to satisfy the flexural force demand is computed first.

[5.7.2.2]  [5.7.3.2]

**Flexural Resistance**

Assume a rectangular stress distribution and solve for the required area of reinforcing based on $M_u$ and $d$. For an $f'_c$ of 4.0 ksi and a $\beta_1$ of 0.85 the equation for the required area of steel reduces to:
Compute “d” values for both a single layer of reinforcement and a double layer of reinforcement. Assume the stirrups are #16 bars, that the primary steel is #32 bars, and that the clear dimension between layers is 1.5 inches. The “d” for a single layer of reinforcement is:

\[ d = 56 - 2 - 0.625 - \frac{1.27}{2} = 52.74 \text{ in} \]

The assumed “d” for two layers of reinforcement is:

\[ d = 56 - 2 - 0.625 - 1.27 - \frac{1.5}{2} = 51.36 \text{ in} \]

Using the Strength I design forces and assuming one layer of reinforcement for the positive moment steel and two layers of reinforcement for the negative steel, the required areas of steel can be found. They are presented in Table 11.4.3.14.

<table>
<thead>
<tr>
<th>Location</th>
<th>Mu(kip-ft)</th>
<th>d</th>
<th>As(req)</th>
<th>Trial Bars</th>
<th>A_s(prov)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Positive Moment</td>
<td>1563</td>
<td>52.74</td>
<td>6.78</td>
<td>6-#32</td>
<td>7.62</td>
</tr>
<tr>
<td>Negative Moment</td>
<td>2337</td>
<td>51.36</td>
<td>10.59</td>
<td>12-#29</td>
<td>12.00</td>
</tr>
</tbody>
</table>

**Crack Control**

To ensure that cracking is limited to small cracks that are well distributed, a limit is placed on the service load stress in the reinforcing steel. LRFD Equation 5.7.3.4-1 defines the maximum stress permitted:
The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement. To simplify the calculations, the section is assumed to be singly reinforced.

\[
fs \leq f_{sa} = \frac{z}{\sqrt[3]{d_c \cdot A}} \leq 0.6 \cdot f_y
\]

Use \( n = 8 \)

\[
n \cdot A_s = 8 \cdot (7.62) = 60.96 \text{ in}^2
\]

Determine the location of the neutral axis:

\[
b \cdot x \cdot \frac{x}{2} = n \cdot A_s \cdot (d - x)
\]

\[
\frac{(40) \cdot x^2}{2} = 60.96 \cdot (52.74 - x)
\]

Solving, \( x = 11.25 \text{ in} \)

Determine the lever arm between service load flexural force components.

\[
j \cdot d = d - \frac{x}{3} = 52.74 - \frac{11.25}{3} = 48.99 \text{ in}
\]

Compute the stress in the reinforcement.

\[
Actual \quad f_s = \frac{M}{A_s \cdot j \cdot d} = \frac{1025 \cdot 12}{7.62 \cdot (48.99)} = 32.9 \text{ ksi}
\]
For \( z = 130 \) kips/in and \( d_c = 2.64 \) in (2.0 in + \( \frac{1}{2} \) of #32 bar), the area of concrete assumed to participate with the reinforcement is:

\[
A = \frac{2 \cdot d_c \cdot b}{N} = \frac{2 \cdot 2.64 \cdot 40}{6} = 35.20 \text{ in}^2
\]

Allowable \( f_s = \frac{z}{\sqrt[3]{d_c \cdot A}} = \frac{130}{\sqrt[3]{2.64 \cdot 35.20}} = 28.7 \text{ ksi} < 0.6 \cdot f_y = 36 \text{ ksi} \)

Allowable \( f_s = 28.7 \text{ ksi} < 32.9 \text{ ksi} \) NO GOOD

Increase the amount of steel provided by the ratio of stresses:

\[
A_s = \frac{32.9}{28.7} \cdot 7.62 = 8.74 \text{ in}^2
\]

Try 7-#32 bars \( (A_s = 8.89 \text{ in}^2) \)

Then:

\[
\begin{align*}
n \cdot A_s &= 71.12 \text{ in}^2 \\
d &= 52.74 \text{ in} \\
x &= 12.03 \text{ in} \\
jd &= 48.73 \text{ in}
\end{align*}
\]

Actual \( f_s = \frac{1025(12)}{8.89(48.73)} = 28.4 \text{ ksi} \)

\( d_c = 2.64 \text{ in} \)

\( A = 30.17 \text{ in}^2 \)

Allowable \( f_s = \frac{130}{\sqrt[3]{2.64 \cdot 30.17}} = 30.2 \text{ ksi} > 28.4 \text{ ksi} \) OK

**Fatigue**

The stress range in the reinforcement is computed and compared against limits to ensure that adequate fatigue resistance is provided.

The unfactored dead load moment in the positive moment region is 459 kip-ft.

The extreme moments on the cross section when fatigue loading is applied are:

- Maximum moment = 459 + 113 = 572 k-ft
- Minimum moment = 459 + (-4) = 455 k-ft

Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:
For the maximum moment:
\[ f_{\text{max}} = \frac{M}{A_s \cdot j \cdot d} = \frac{572.12}{8.89 \cdot (48.73)} = 15.8 \text{ ksi} \]

For the minimum moment:
\[ f_{\text{min}} = \frac{M}{A_s \cdot j \cdot d} = \frac{455.12}{8.89 \cdot (48.73)} = 12.6 \text{ ksi} \]

The stress range in the reinforcement \((f_f)\) is the difference between the two stresses
\[ f_f = (15.8 - 12.6) = 3.2 \text{ ksi} \]

The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.
\[ f_{f(\text{max})} = 21 - 0.33 \cdot f_{\text{min}} + \frac{f}{h} = 21 - 0.33 \cdot (12.6) + 8 \cdot (0.3) \]
\[ f_{f(\text{max})} = 19.2 > 3.2 \text{ ksi} \quad \text{OK} \]

Check Maximum Reinforcement
To ensure that the cross section is not over-reinforced, the depth of the section in compression is compared to the distance to the tension reinforcement.

The depth of the Whitney stress block is:
\[ a = \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} = \frac{8.89 \cdot 60}{0.85 \cdot 4.0 \cdot 40} = 3.92 \text{ in} \]

The depth of the section in compression is:
\[ c = \frac{a}{\beta_1} = \frac{3.92}{0.85} = 4.61 \text{ in} \]
\[ \frac{c}{d_e} = \frac{4.61}{52.74} = 0.087 < 0.42 \quad \text{OK} \]

Check Minimum Reinforcement
To prevent a brittle failure mode, adequate flexural reinforcement needs to be placed in the cross section. The assumed cracking stress of concrete is:
\[ f_r = 0.24 \cdot \sqrt{f_c} = 0.24 \cdot \sqrt[4]{4} = 0.48 \text{ ksi} \]

The gross moment of inertia is:
\[ I_g = \frac{1}{12} \cdot b \cdot t^3 = \frac{1}{12} \cdot 40 \cdot (56)^3 = 585,400 \text{ in}^4 \]
The distance from the centroid to the tension face is:

\[ y_t = 28.0 \text{ in} \]

Combining these terms to determine the cracking moment produces:

\[ M_{cr} = \frac{f_r' \cdot I_g}{y_t} = \frac{0.48 \cdot 585,400}{28.0 \cdot (12)} = 836 \text{ kip-ft} \]

To ensure ductility after cracking moment is reached, the force is increased by 20 percent:

\[ 1.2M_{cr} = 1003 \text{ kip-ft} \]

The capacity of the steel provided is:

\[ M_r = \phi A_s f_y (d - a/2) \]

Which after substituting values becomes:

\[ M_r = 0.9 (8.89) \cdot (60) \left( \frac{52.74 - 3.92}{2} \right) \cdot \frac{1}{12} = 2031 \text{ kip-ft} > 1003 \text{ kip-ft} \text{ OK} \]

Provide 1 layer of 7 - #32 bars for positive reinforcement.

### 3. Design Negative Moment Reinforcement

The service, and fatigue checks for the negative moment steel are performed next.

**Flexural Resistance**

The required area of steel to satisfy the strength check was presented in Table 11.4.3.14. Try 2 layers of 6-#29 bars \((A_s = 12.00 \text{ in}^2)\) with a clear spacing between layers equal to 1.5 inches.

\[ d = 56 - 2 - 0.625 - 1.128 - \frac{1.5}{2} = 51.50 \text{ in} \]

**[5.7.3.4]**

**Crack Control**

The stress in the reinforcement is found using a cracked section analysis with the trial reinforcement.

The transformed area of steel is:

\[ n \cdot A_s = 8 \cdot (12.00) = 96.0 \text{ in}^2 \]

The location of the neutral axis satisfies:

\[ \frac{(40) \cdot x^2}{2} = 96.0 \cdot (51.50 - x) \text{ solving, } x = 13.50 \text{ in} \]
The lever arm between service load flexural force components is:

\[ j \cdot d = d - \frac{x}{3} = 51.50 - \frac{13.50}{3} = 47.00 \text{ in} \]

And the stress in the reinforcement is:

Actual \[ f_s = \frac{M}{A_s \cdot j \cdot d} = \frac{1655 \cdot 12}{12.00 \cdot (47.00)} = 35.2 \text{ ksi} \]

For a \( z \) of 130 kips/in and \( d_c = 2.56 \text{ in} \) (2.0 in + \( \frac{1}{2} \) of #29 bar),

\[ A = \frac{2 \cdot [2 + 1.128 + 0.75] \cdot b}{N} = \frac{2 \cdot 3.878 \cdot 40}{12} = 25.85 \text{ in}^2 \]

The permitted stress in the reinforcement is:

Allowable \[ f_s = \frac{z}{\sqrt[3]{d_c \cdot A}} = \frac{130}{\sqrt[3]{2.56 \cdot 25.85}} = 32.1 \text{ ksi} < 0.6 \cdot f_y = 36 \text{ ksi} \]

Allowable \( f_s = 32.1 \text{ ksi} < 35.2 \text{ ksi} \) \( \text{NO GOOD} \)

The actual stress is larger than the permitted. Increase the amount of reinforcement by the ratio of the stresses:

\[ A_s = \frac{35.2}{32.1} \cdot 12.00 = 13.16 \text{ in}^2 \]

Try 2 layers of 6-#32 bars (\( A_s = 15.24 \text{ in}^2 \))

\[ n \cdot A_s = 121.92 \text{ in}^2 \]
\[ d = 15.36 \text{ in} \]
\[ x = 14.91 \text{ in} \]
\[ j d = 46.39 \text{ in} \]

Actual \[ f_s = \frac{1655 \cdot 12}{15.24 \cdot 46.39} = 28.1 \text{ ksi} \]

\[ d_c = 2.64 \]
\[ A = 26.8 \text{ in}^2 \]

Allowable \[ f_s = \frac{130}{\sqrt[3]{2.64 \cdot 26.8}} = 31.4 \text{ ksi} > 28.1 \text{ ksi} \] \( \text{OK} \)

[5.5.3]

**Fatigue**

The moments on the negative moment section when fatigue loading is applied vary from:

- Maximum moment = 1118 + 125 = 1243 k-ft
- Minimum moment = 1118 + 0 = 1118 k-ft
Plugging these moments into the equation used to compute the stress in the reinforcement for crack control results in:

For the maximum moment:

\[ f_{\text{max}} = \frac{M}{A_s \cdot j \cdot d} = \frac{1243 \cdot 12}{15.24 \cdot (46.39)} = 21.1 \text{ ksi} \]

For the minimum moment:

\[ f_{\text{min}} = \frac{M}{A_s \cdot j \cdot d} = \frac{1118 \cdot 12}{15.24 \cdot (46.39)} = 19.0 \text{ ksi} \]

The stress range in the reinforcement \( (f_r) \) is the difference between the two stresses

\[ f_r = (21.1 - 19.0) = 2.1 \text{ ksi} \]

The maximum stress range permitted is based on the minimum stress in the bar and the deformation pattern of the reinforcement.

\[ f_{r(\text{max})} = 21 - 0.33 \cdot f_{\text{min}} + 8 \cdot \frac{r}{h} = 21 - 0.33 \cdot (19.0) + 8 \cdot (0.3) = 17.1 \text{ ksi} \]

\[ f_{f(\text{max})} = 17.1 > 2.1 \text{ ksi} \quad \text{OK} \]

[5.5.3.2] Check Maximum Reinforcement

The depth of the Whitney stress block is:

\[ a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{15.24 \cdot 60}{0.85 \cdot 4.0 \cdot 40} = 6.72 \text{ in} \]

The depth of the section in compression is:

\[ c = \frac{a}{\beta_1} = \frac{6.72}{0.85} = 7.91 \text{ in} \]

\[ \frac{c}{d} = \frac{7.91}{51.36} = 0.15 < 0.42 \quad \text{OK} \]

[5.7.3.3.2] Check Minimum Reinforcement

\[ 1.2M_{cr} = 1003 \text{ kip-ft} \]

The moment capacity provided is:

\[ M_r = \phi A_s f_y (d - a/2) = 0.9 \cdot (15.24) \cdot (60) \cdot \left[ \frac{51.36 - 6.72}{2} \right] \cdot \frac{1}{12} \]

\[ = 3292 \text{ kip-ft} > 1003 \text{ kip-ft} \quad \text{OK} \]

Provide 2 layers of 6-#32 bars for negative moment reinforcement.
4. Design Shear Reinforcement
The maximum factored design shear force is 722 kips (Strength I for Live Load Case 4) and occurs at the centerline of Column 2.

\[ V_n = \frac{V_u}{\phi_v} = \frac{722}{0.90} = 802 \text{ kips} \]

The shear design for reinforced concrete elements is a two step process. First, the shear capacity of the concrete section is determined. Second, the amount of shear steel is determined. The concrete capacity is dependent on \( \theta \), the angle of inclination of the concrete struts, and \( \beta \), a factor indicating the ability of the diagonally cracked concrete to transmit tension.

[5.8.3.4.1] Determine Concrete Shear Capacity
The minimum shear reinforcement will be provided in the section.

Therefore, \( \beta = 2.0 \) and \( \theta = 45 \) degrees

\( d_v \) is the distance between the internal flexural force components. The smaller distance between the “C” and “T” centroids is for the negative moment steel:

\[ d_v = \frac{a}{2} = \frac{6.72}{2} = 48.0 \text{ in} \]

[5.8.2.9] However, \( d_v \) need not be less than:

\[ 0.72 \cdot h = 0.72 \cdot 56 = 40.3 \text{ in} \]

or

\[ 0.90 \cdot d = 0.90 \cdot 51.36 = 46.2 \text{ in} \]

Use \( d_v = 48.0 \) in

[5.8.3.3-3] With \( d_v \) known, the concrete shear capacity can be computed:

\[ V_c = 0.0316 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v = 0.0316 \cdot 2 \cdot \sqrt{4} \cdot 48 \cdot 40 = 243 \text{ kips} \]

Determine Stirrup Spacing
The difference between the required shear capacity and the capacity provided by the concrete is the required capacity for the shear steel.

\[ V_s = V_n - V_c = 802 - 243 = 559 \text{ kips} \]

Use \#16 double "U" stirrups that will be vertical. Four legs of \#16 bars have an area of:

\[ A_v = 4 \cdot A_b = 4 \cdot 0.31 = 1.24 \text{ in}^2 \]
The capacity of shear steel is:

\[ V_s = \frac{A_v \cdot f_v \cdot d_v \cdot \cot(\theta)}{s} \]

Which can be rearranged to solve for the stirrup spacing:

\[ s \leq \frac{A_v \cdot f_v \cdot d_v \cdot \cot(\theta)}{V_s} = \frac{[1.24 \cdot 60 \cdot 48.0 \cdot 1]}{559} = 6.4 \text{ in} \]

To simplify construction, try a constant stirrup spacing of 6.0 inches between columns and in pier cap cantilever.

Check Minimum Shear Reinforcement Requirements

Determine maximum stirrup spacing that satisfies minimum transverse reinforcement requirements:

\[ A_v \geq 0.0316 \cdot \sqrt{f'_c} \cdot \frac{b_v \cdot s}{f_y} \]

Rearranging and solving for stirrup spacing \( s \),

\[ s \leq \frac{A_v \cdot f_y}{0.0316 \cdot b_v \cdot \sqrt{f'_c}} = \frac{1.24 \cdot 60}{0.0316 \cdot 40 \cdot \sqrt{4}} = 29.4 \text{ in} >> 6 \text{ in} \quad \text{OK} \]

Check Maximum Shear Reinforcement Spacing Requirements

First determine if \( v_u < 0.125f'_c \):

\[ 0.125f'_c = 0.125(4) = 0.50 \text{ ksi} \]

\[ v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{722 - 0}{0.9 \cdot 40 \cdot 48.0} = 0.42 \text{ ksi} < 0.50 \text{ ksi} \]

Therefore,

\[ S_{\text{max}} = 0.8d_v = 0.8 \cdot 48 = 38.4 \text{ in} \]

or \( S_{\text{max}} = 24.0 \text{ in} \quad \text{GOVERNS} \)

\[ S_{\text{max}} = 24.0 \text{ in} >> 6 \text{ in} \quad \text{OK} \]

Use #16 double “U” stirrups at a 6 inch spacing for shear reinforcement in the pier cap.

5. Cantilever Capacity Check

Check the capacity of the cantilever using strut and tie methods. Begin by determining the vertical reaction applied to the cantilever.
Self weight of cantilever:

\[
P_{\text{self}} = \left( \frac{40}{12} \right) \cdot \left( \frac{56 + 36}{2} \right) \cdot \left( \frac{1}{12} \right) \cdot 5.25 \cdot 0.150 = 10.0 \text{ kips}
\]

Dead load from the superstructure is:

\[P_{\text{super}} = 28.4 \text{ kips}\]

The reaction from one lane of live load is:

\[P_U = 143.1 \text{ kips} \quad (\text{LL Case 3})\]

Then the factored vertical load on the cantilever is:

\[1.25 \cdot (10.0 + 284) \cdot 1.75 \cdot (143.1) = 618 \text{ kips}\]

Assume a simple model with a single horizontal tension tie centered on the top reinforcement and a single compression strut between the center of the tension tie below the bearing and the center of the column. A simple schematic with the resultant loads in the strut and tie is shown in Figure 11.4.3.8.

\[\text{Cantilever Strut and Tie Model}\]

\[\text{Figure 11.4.3.8}\]

**Tension Tie**

The required capacity of the tension tie is:

\[
\frac{T}{\phi} = \frac{537}{0.9} = 597 \text{ kips}
\]
The tie is composed of 12-#32 bars ($A_s = 15.24 \text{ in}^2$). With standard hooks at the ends, the bars are developed by the time they reach the intersection of the tie and strut. Then the capacity of the tie is:

\[ P_n = f_y \cdot A_{st} = 60 \cdot 15.24 = 914 \text{ kips} \gg 597 \text{ kips} \quad \text{OK} \]

**Concrete Strut**

The required capacity of the concrete compression strut is:

\[ C = \frac{819}{0.70} = 1170 \text{ kips} \]

The width of the strut (measured along direction parallel to pier) is chosen based on the width of the bearing pads and adjusted for the angle of the strut. The bearing pads are 24 inches wide and the strut is inclined at 49 degrees from horizontal. Then the width of the strut is:

\[ W = 24 \cdot \sin 49^\circ = 18.1 \text{ in} \]

Using the pier cap thickness of 40 inches for the thickness of the strut, the cross-sectional area of the strut is:

\[ A_{cs} = W \cdot T = 18.1 \cdot 40.0 = 724 \text{ in}^2 \]

**[C5.6.3.3.3]**

The allowable compressive stress in the strut is dependent on the strain in the tension ties crossing the strut. The strain in the tension tie is found assuming a cracked cross section.

\[ \sigma = \frac{537}{15.24} = 35.2 \text{ ksi} \]

The strain in the tie is:

\[ \varepsilon_s = \frac{\sigma}{E} = \frac{35.2}{29,000} = 0.00121 \]

The concrete strain component is:

\[ \varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002) \cdot \cot^2 \alpha \]

\[ = 0.00121 + (0.00121 + 0.002) \cdot \cot^2 (49^\circ) = 0.00364 \]

The limiting compressive stress in the strut is:

\[ f_{cu} = \frac{f'_c}{0.8 + 170 \cdot \varepsilon_1} = \frac{4}{0.8 + 170 \cdot 0.00364} = 2.82 \text{ ksi} \]

**[5.6.3.3.1]**

The capacity of the strut is:

\[ P_n = A_{cs} \cdot f_{cu} = 724 \cdot 2.82 = 2042 \text{ kips} \gg 1170 \text{ kips} \quad \text{OK} \]
[5.6.3.5] Check the Node Region
The concrete compressive stress in node regions needs to be examined to ensure that stresses are below acceptable limits. For node regions anchoring a one-direction tension tie, the limit on compressive stresses is:

$$0.75 \cdot \phi \cdot f'_c = 0.75 \cdot 0.70 \cdot 4.0 = 2.10 \text{ ksi}$$

By inspection, the node section on the concrete strut side governs. The compressive stress on the node is:

$$\frac{P}{A_{cs}} = \frac{819}{724} = 1.13 \text{ ksi} << 2.10 \text{ ksi} \quad \text{OK}$$

[5.7.3.4] 6. Longitudinal Skin Reinforcement
The effective depth for both positive and negative moment reinforcement is greater than 3.0 feet, so skin reinforcement is required. The minimum area of skin reinforcement required on each vertical face of the pier cap is:

Positive moment region:

$$A_{sk} \geq 0.012(d_e - 30) = 0.012(52.74 - 30) = 0.27 \text{ in}^2/\text{ft}$$

but not more than $$A_{sk} \leq \frac{A_s}{4} = \frac{8.89}{4} = 2.22 \text{ in}^2/\text{ft}$$

Negative moment region:

$$A_{sk} \geq 0.012 (51.36 - 30) = 0.26 \text{ in}^2/\text{ft}$$

but not more than $$A_{sk} \leq \frac{15.24}{4} = 3.81 \text{ in}^2/\text{ft}$$

The skin reinforcement must be placed within $d/2$ of the main reinforcement with a spacing not to exceed $d/6$ or 12 inches.

Using the smallest $d =

$$\frac{d}{2} = \frac{51.36}{2} = 25.68 \text{ in}$$

$$\frac{d}{6} = \frac{51.36}{6} = 8.56 \text{ in}$$

Choose 5-#16 bars equally spaced between the top and bottom reinforcement on each face. (Spacing = 7.79 in and $A_s = 0.48 \text{ in}^2/\text{ft}$)

[5.10.8.2-1] 7. Temperature Steel Check
A minimum amount of longitudinal steel needs to be provided on the vertical faces to ensure that shrinkage and temperature cracks remain
small and well distributed. The minimum amount of steel to be provided is:

\[
\text{Min. total } A_s \geq 0.11 \cdot \frac{A_g}{f_y} = 0.11 \cdot \frac{(40 \cdot 56)}{60} = 4.11 \text{in}^2
\]

The actual total reinforcement area for both vertical faces is:

Actual total \( A_s = 2 \cdot [3 \cdot (1.27) + 5 \cdot (0.31)] = 10.72 \text{ in}^2 > 4.11 \text{ in}^2 \quad \text{OK}

8. Summary

Figure 11.4.3.9 details the final reinforcement in the pier cap.
Pier Cap Reinforcement

Figure 11.4.3.9
Design Forces

Table 11.4.3.15 lists the unfactored axial loads and bending moments at the top and bottom of the columns when the pier is subjected to various loadings.

The sign convention for the axial loads is positive for downward forces and negative for upward forces. The sign convention for the bending moments in the parallel direction ($M_{\text{par}}$) is beam convention. Positive moments cause tension on the “bottom side” of the column member which is defined as the right side of the column. Negative moments cause tension on the “top side” which is defined as the left side. (See Figure 11.4.3.10.)

**Sign Convention for $M_{\text{par}}$**

*Figure 11.4.3.10*

For moments in the perpendicular direction ($M_{\text{perp}}$), all lateral loads are assumed applied in the same direction. Therefore, all moments are shown as positive.

Moments shown in the table due to wind transverse to the bridge are based on a wind directed from right to left. (Column 3 is on the windward side of the pier.)
<table>
<thead>
<tr>
<th>Load</th>
<th>Force</th>
<th>Column 1 (Leeward)</th>
<th>Column 2 (Center)</th>
<th>Column 3 (Windward)</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Top</td>
<td>Bottom</td>
<td>Top</td>
<td>Bottom</td>
</tr>
<tr>
<td>Dead Load</td>
<td>P</td>
<td>608</td>
<td>629</td>
<td>663</td>
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<tr>
<td></td>
<td>Bottom</td>
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<td>608</td>
<td>629</td>
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<td>4.3</td>
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<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
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<td>0</td>
<td>0</td>
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<td>Live Load Case 1 One Lane</td>
<td>P</td>
<td>108</td>
<td>108</td>
<td>137</td>
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<td></td>
<td>Bottom</td>
<td>137</td>
<td>137</td>
<td>10-10</td>
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<td>0</td>
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<td>Live Load Cases 2 and 3</td>
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<td>233/19</td>
<td>233/19</td>
<td>196/9</td>
</tr>
<tr>
<td>One Lane (max/min)</td>
<td>Bottom</td>
<td>196/9</td>
<td>196/9</td>
<td>19/-5</td>
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<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>47/-40</td>
<td>19/-12</td>
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<td>188/47</td>
<td>297/213</td>
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<td>Two Lanes (max/min)</td>
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<td>297/213</td>
<td>47/-8</td>
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<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>35° F Temperature Rise</td>
<td>P</td>
<td>7</td>
<td>7</td>
<td>-14</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>-14</td>
<td>-14</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>-100</td>
<td>110</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wind at 0° on Superstructure</td>
<td>P</td>
<td>36</td>
<td>36</td>
<td>0</td>
</tr>
<tr>
<td>and Substructure</td>
<td>Bottom</td>
<td>0</td>
<td>0</td>
<td>-36</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>-249</td>
<td>270</td>
<td>-279</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wind at 30° on Superstructure</td>
<td>P</td>
<td>30</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>and Substructure</td>
<td>Bottom</td>
<td>0</td>
<td>0</td>
<td>-29</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>-204</td>
<td>221</td>
<td>-229</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wind at 60° on Superstructure</td>
<td>P</td>
<td>12</td>
<td>12</td>
<td>0</td>
</tr>
<tr>
<td>and Substructure</td>
<td>Bottom</td>
<td>0</td>
<td>0</td>
<td>-12</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>-86</td>
<td>93</td>
<td>-96</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>294</td>
<td>0</td>
</tr>
<tr>
<td>Vertical Wind</td>
<td>P</td>
<td>2</td>
<td>2</td>
<td>-46</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>-46</td>
<td>-46</td>
<td>-89</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wind on Live Load at 0°</td>
<td>P</td>
<td>11</td>
<td>11</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>0</td>
<td>0</td>
<td>-11</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>-50</td>
<td>55</td>
<td>-56</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wind on Live Load at 30°</td>
<td>P</td>
<td>9</td>
<td>9</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>0</td>
<td>0</td>
<td>-9</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>-41</td>
<td>44</td>
<td>-46</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>29</td>
<td>0</td>
</tr>
<tr>
<td>Wind on Live Load at 60°</td>
<td>P</td>
<td>4</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>0</td>
<td>0</td>
<td>-4</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;par&lt;/sub&gt;</td>
<td>-17</td>
<td>18</td>
<td>-19</td>
</tr>
<tr>
<td></td>
<td>M&lt;sub&gt;perp&lt;/sub&gt;</td>
<td>0</td>
<td>46</td>
<td>0</td>
</tr>
</tbody>
</table>
The following four limit states are examined for the columns:

Strength I: \[ U_1 = \gamma_p \cdot DC + 1.75 \cdot LL + 1.75 \cdot BR + 0.50 \cdot TU \]

Strength III: \[ U_3 = \gamma_p \cdot DC + 1.40 \cdot WS + 0.50 \cdot TU \]

Strength IV: \[ U_3 = 1.50 \cdot DC + 0.50 \cdot TU \]

Strength V:
\[ U_5 = \gamma_p \cdot DC + 1.35 \cdot LL + 1.35 \cdot BR + 0.40 \cdot WS + 1.00 \cdot WL + 0.50 \cdot TU \]

Load combinations were tabulated for the appropriate limit states for each of the various live load cases, wind angles, the temperature rise and fall, and also for maximum and minimum DC load factors.

Then the worst case loadings (maximum axial load with maximum moment, minimum axial load with maximum moment) were chosen from each limit state from the tabulated load combinations. These are shown in Table 11.4.3.16. The critical cases for the column among those listed in the table are shown in bold print.

### Table 11.4.3.16 - Column Design Forces

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial Load P (kips)</th>
<th>( M_{par} ) (kip-ft)</th>
<th>( M_{perp} ) (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strength I:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Column 2 Bottom: ( \gamma_p = 1.25, ) LL Case 6, ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>1396</td>
<td>0</td>
<td>597</td>
</tr>
<tr>
<td>(b) Column 3 Bottom: ( \gamma_p = 0.90, ) LL Case 1, ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>544</td>
<td>55</td>
<td>597</td>
</tr>
<tr>
<td><strong>Strength III:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Column 2 Bottom: ( \gamma_p = 1.25, ) Wind Skew = 60°, ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>864</td>
<td>137</td>
<td>413</td>
</tr>
<tr>
<td>(b) Column 3 Bottom: ( \gamma_p = 0.90, ) Wind Skew = 60°, ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>545</td>
<td>200</td>
<td>412</td>
</tr>
<tr>
<td>(c) Column 3 Bottom: ( \gamma_p = 0.90, ) Wind Skew = 0°, Vertical Wind, ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>387</td>
<td>454</td>
<td>0</td>
</tr>
<tr>
<td><strong>Strength IV:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Column 1 Bottom: ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>939</td>
<td>64</td>
<td>0</td>
</tr>
<tr>
<td>(b) Column 2 Bottom: ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>1035</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>Strength V:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) Column 2 Bottom: ( \gamma_p = 1.25, ) LL Case 6, Wind Skew = 60°, ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>1274</td>
<td>58</td>
<td>624</td>
</tr>
<tr>
<td>(b) Column 3 Bottom: ( \gamma_p = 0.90, ) LL Case 1, Wind Skew = 60°, ( \Delta \text{Temp} = -45^\circ \text{F} )</td>
<td>539</td>
<td>114</td>
<td>624</td>
</tr>
</tbody>
</table>
**Slenderness Effects**

Each column is considered unbraced in both the parallel and perpendicular directions. The dimension “L” from bottom of pier cap to top of footing is 17.58 feet.

In the parallel direction, a fixed condition exists at the bottom and a rotation-fixed, translation-free condition exists at the top. For this condition LRFD Table C4.6.2.5-1 recommends a K value of 1.20.

Then:
\[
\frac{KL}{r} = 0.25 \cdot \text{(column diameter)} = 0.25 \cdot (3) = 0.75 \text{ ft}
\]

\[
\frac{1.2(17.58)}{0.75} = 28.1 > 22
\]

Therefore, slenderness effects need to be considered for the parallel direction.

In the perpendicular direction the columns can conservatively be considered as cantilevers fixed at the bottom. For this condition LRFD Table C4.6.2.5-1 recommends a K value of 2.1.

Then:
\[
\frac{KL}{r} = \frac{2.1(17.58)}{0.75} = 49.2 > 22
\]

Therefore, slenderness effects need to be considered for the perpendicular direction, also.

Two choices are available to designers when including slenderness effects in the design of columns. A moment magnification method is described in LRFD Article 4.5.3.2.2. The other method is to use an iterative P-Δ analysis.

A P-Δ analysis was used for this example. For simplicity and in order to better match the computer model used, take the column height L equal to the distance from the top of footing to the centroid of the pier cap.

For the perpendicular direction, the maximum factored moment and corresponding axial load from Table 11.4.3.16 is:

\[
M_{\text{perp}} = 624 \text{ kip-ft, } P = 1274 \text{ kips (Strength V (a))}
\]

Then the maximum equivalent lateral force \(H_{\text{perp}}\) applied at the top of the column is:

\[
H_{\text{perp}} = \frac{M_{\text{perp}}}{L} = \frac{624}{19.92} = 31.3 \text{ kips}
\]
This force produces a perpendicular displacement $\Delta_{\text{perp}}$ at the top of the column:

$$
\Delta_{\text{perp}} = \frac{H_{\text{perp}} L^3}{3EI} = \frac{31.3 \cdot [(19.92)(12)]^3}{3 \cdot (3644) \cdot (82450)} = 0.474 \text{ in}
$$

The structural model used in the analysis contained gross section properties. To account for the reduced stiffness of a cracked column section, the displacement was multiplied by an assumed cracked section factor equal to 2.5. This factor is based on using LRFD Equation 5.7.4.3-2 with $\beta_d$ equal to zero and corresponds to 40 percent of the gross section properties being effective. (Other references suggest values ranging from 30 percent to 70 percent be used for columns.) After updating the equivalent lateral force for the P-\(\Delta\) moment, three additional iterations were performed. The final longitudinal displacement was found to be 0.593 inches and the additional perpendicular moment due to slenderness was 157 kip-feet. See Figure 11.4.3.11 and Table 11.4.3.17.

For the parallel direction, the corresponding factored moment from Table 11.4.3.16 is:

$$
M_{\text{par}} = 58 \text{ kip} \cdot \text{ft} \quad \text{(Strength V (a))}
$$

A procedure similar to that done for the perpendicular direction was used for the P-\(\Delta\) analysis. For the parallel direction, equations used to compute $H_{\text{par}}$ and $\Delta_{\text{par}}$ are for a cantilever column fixed at one end and free to deflect horizontally but not rotate at the other end (taken from “Manual of Steel Construction, Allowable Stress Design, Ninth Edition”, page 2-303). See Figure 11.4.3.12 and Table 11.4.3.18 for a summary of the parallel direction P-\(\Delta\) analysis.

This process was repeated for the other three critical load cases shown in Table 11.4.3.16.
Perpendicular Direction $P-\Delta$ Procedure

*Figure 11.4.3.11*
### Table 11.4.3.17 – Perpendicular P-Δ Moment

<table>
<thead>
<tr>
<th>Equiv. Lateral Force H (kips/column)</th>
<th>Axial Load P (kips/column)</th>
<th>$\Delta g$ for gross section properties (in)</th>
<th>Cracked Section Factor $F_{cr}$</th>
<th>$\Delta cr$ for cracked section (in)</th>
<th>$M_{PA}$ (k-ft)</th>
<th>$\Delta H$ to produce $M_{PA}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>31.3</td>
<td>1274</td>
<td>0.474</td>
<td>2.5</td>
<td>0.948</td>
<td>100.6</td>
<td>5.1</td>
</tr>
<tr>
<td>36.4</td>
<td>1274</td>
<td>0.552</td>
<td>2.5</td>
<td>1.380</td>
<td>146.5</td>
<td>7.4</td>
</tr>
<tr>
<td>38.7</td>
<td>1274</td>
<td>0.586</td>
<td>2.5</td>
<td>1.465</td>
<td>155.5</td>
<td>7.8</td>
</tr>
<tr>
<td>39.1</td>
<td>1274</td>
<td>0.593</td>
<td>2.5</td>
<td>1.483</td>
<td>157.4</td>
<td></td>
</tr>
</tbody>
</table>

Add 157 k-ft to column for slenderness in the perpendicular direction

### Parallel Direction P-Δ Procedure

*Figure 11.4.3.12*
Table 11.4.3.18 – Parallel P-Δ Moment

<table>
<thead>
<tr>
<th>Equiv. Lateral Force H (kips/column)</th>
<th>Axial Load P (kips/column)</th>
<th>( \Delta g ) for gross section properties (in)</th>
<th>Cracked Section Factor ( F_{cr} )</th>
<th>( \Delta_{cr} ) cracked section (in)</th>
<th>( M_{PA} ) (kips)</th>
<th>( \Delta H ) to produce ( M_{PA} ) (k-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.82</td>
<td>1274</td>
<td>0.0220</td>
<td>2.5</td>
<td>0.0500</td>
<td>5.3</td>
<td>0.53</td>
</tr>
<tr>
<td>6.35</td>
<td>1274</td>
<td>0.0241</td>
<td>2.5</td>
<td>0.0603</td>
<td>6.4</td>
<td>0.64</td>
</tr>
<tr>
<td>6.46</td>
<td>1274</td>
<td>0.0245</td>
<td>2.5</td>
<td>0.0613</td>
<td>6.5</td>
<td></td>
</tr>
</tbody>
</table>

Add 7 k-ft to column for slenderness in the parallel direction.

The design forces presented in Table 11.4.3.19 are the factored axial loads and resultant moments that include P-Δ effects. Because of the symmetry of the round cross section, the moments in the parallel and perpendicular directions can be combined using the square root of the sum of the squares (Pythagorean Theorem).

\[ M_R = \sqrt{M_{par}^2 + M_{perp}^2} \]

Table 11.4.3.19 – Critical Column Design Forces (kips, kip-ft)

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial Load</th>
<th>( M_{par} )</th>
<th>( M_{par} ) P-Δ</th>
<th>Total ( M_{par} )</th>
<th>( M_{perp} )</th>
<th>( M_{perp} ) P-Δ</th>
<th>Total ( M_{perp} )</th>
<th>Resultant ( M_R )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I (a)</td>
<td>1396</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>597</td>
<td>169</td>
<td>766</td>
<td>766</td>
</tr>
<tr>
<td>Strength III (c)</td>
<td>387</td>
<td>454</td>
<td>14</td>
<td>468</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>468</td>
</tr>
<tr>
<td>Strength V (a)</td>
<td>1274</td>
<td>58</td>
<td>7</td>
<td>65</td>
<td>624</td>
<td>157</td>
<td>781</td>
<td>784</td>
</tr>
<tr>
<td>Strength V (b)</td>
<td>539</td>
<td>114</td>
<td>5</td>
<td>119</td>
<td>624</td>
<td>58</td>
<td>682</td>
<td>692</td>
</tr>
</tbody>
</table>

The minimum amount of column reinforcement must be such that:

\[ A_s f_y \geq 0.135 \]
\[ A_g f_c \]

Then:

\[ \text{Min } A_s \geq \left( \frac{A_g f_c}{f_y} \right) \cdot 0.135 = \left( \frac{1018 \cdot 4.0}{60.0} \right) \cdot 0.135 = 9.16 \text{ in}^2 \]

Try 12-#25 bars (\( A_s = 9.48 \text{ in}^2 \)).

A computer program was used to generate the column strength interaction diagram shown in Figure 11.4.3.13. The figure also displays...
the design axial loads and moments for the critical load cases. All values fall well within the capacity of the column.

The interaction diagram includes $\phi$ factors of 0.90 for flexure and 0.75 for axial compression.

![COLUMN INTERACTION CURVE](image)

**Figure 11.4.3.13**

[5.7.4.2] **Reinforcement Limit Check**
For non-prestressed columns the maximum amount of longitudinal reinforcement permitted is:

$$\frac{A_s}{A_g} = \frac{9.48}{1018} = 0.00931 \leq 0.08 \quad \text{OK}$$
[5.10.6.2] **Column Spirals**
Per Mn/DOT standard practice, use spiral reinforcing for columns with diameters up to 42". Use #13E bars with a 3" pitch for the spiral. The anchorage of the spiral reinforcement shall be provided by 1½ extra turns of spiral bar at each end of the spiral unit.

[5.7.4.6] Check reinforcement ratio of spiral to concrete core:

\[ p_s = \frac{\text{volume of spiral in one loop}}{\text{volume of core for one pitch spacing}} \]

For a clear cover of 2", diameter of the core \( D_c = 32 \) in.

Spiral reinforcement area \( A_{sp} = 0.20 \text{ in}^2 \)

Spiral bar diameter \( d_b = 0.50 \) in

Pitch spacing \( p = 3.0 \) in

Length of one loop

\[ \ell_{sp} = \sqrt{\left[ \pi \cdot (D_c - d_b) \right]^2 + p^2} = \sqrt{\left[ \pi \cdot (32 - 0.50) \right]^2 + (3.0)^2} = 99.01 \text{ in} \]

Then Actual

\[ p_s = \frac{A_{sp} \cdot \ell_{sp}}{\left( \frac{\pi \cdot D_c^2}{4} \right) \cdot p} = \frac{0.20 \cdot 99.01}{\left( \frac{\pi \cdot 32^2}{4} \right) \cdot 3.0} = 0.00821 \]

Req’d. Min.

\[ p_s = 0.45 \cdot \left( \frac{A_g}{A_c} - 1 \right) \cdot \frac{f_{c'}^*}{f_{yh}} \]

\[ = 0.45 \cdot \left( \frac{1018}{\left( \frac{\pi \cdot D_c^2}{4} \right)} - 1 \right) \cdot \frac{4}{60} = 0.00797 < 0.00821 \quad \text{OK} \]

**F. Design Piling**

**Loads**
For design of the piles and footings, additional loads which include the weight of the footing and an assumed 2'-0" of earth must be added to the critical column design forces in Table 11.4.3.19.
Additional DC due to footing:
\[ P = 10.0 \cdot 13.0 \cdot 4.50 \cdot 0.150 = 87.8 \text{ kips} \]
\[ M_{\text{par}} = M_{\text{perp}} = 0 \text{ kip-ft} \]

Earth above footing EV:
\[ P = 2.0 \left( 10 \cdot 13 - \frac{1018}{144} \right) \cdot 0.120 = 29.5 \text{ kips} \]
\[ M_{\text{par}} = M_{\text{perp}} = 0 \text{ kip-ft} \]

Use a maximum load factor of 1.35 and a minimum load factor of 0.90. Also, the dynamic load allowance is to be removed from the live load when designing foundation components entirely below ground.

For one live load lane (Live Load Case 1, Column 3):
Single Lane Truck Reaction w/Dynamic Load Allowance, \( R=134.1 \text{ kips} \)
(From Table 11.4.3.6)
Then
\[ P_{\text{red}} = \left( R - \frac{R}{\text{DLA}} \right) \cdot (\text{Mult. Presence Factor}) \cdot (\text{Double Truck Load F.}) \cdot \left( \frac{1}{\text{No. Columns}} \right) \]
\[ = \left( 134.1 - \frac{134.1}{1.33} \right) \cdot (1.2) \cdot (0.90) \cdot \left( \frac{1}{3} \right) = 12.0 \text{ kips} \]

The reduction to \( M_{\text{par}} \) is negligible and will be ignored.

For three live load lanes (Live Load Case 6, Column 2):
\[ P_{\text{red}} = \left( 134.1 - \frac{134.1}{1.33} \right) \cdot (0.85) \cdot (0.90) \cdot \left( \frac{1}{3} \right) = 8.5 \text{ kips} \]

Then, for example, the Strength I(a) piling design forces are:
Axial Load = 1396 + 1.25 \cdot (87.8) + 1.35 \cdot (29.5) - 1.75 \cdot (8.5) = 1531 \text{ kips}
\[ M_{\text{par}} = 0 \text{ kip-ft} \]
\[ M_{\text{perp}} = 766 \text{ kip-ft} \]

The values for all of the critical piling forces are shown in Table 11.4.3.21.
Table 11.4.3.21 – Piling Design Forces

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial Load (kips)</th>
<th>M&lt;sub&gt;par&lt;/sub&gt; Bending Moment (kip-ft)</th>
<th>M&lt;sub&gt;perp&lt;/sub&gt; Bending Moment (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I (a)</td>
<td>1531</td>
<td>0</td>
<td>766</td>
</tr>
<tr>
<td>Strength III (c)</td>
<td>493</td>
<td>468</td>
<td>0</td>
</tr>
<tr>
<td>Strength V (a)</td>
<td>1412</td>
<td>65</td>
<td>781</td>
</tr>
<tr>
<td>Strength V (b)</td>
<td>628</td>
<td>119</td>
<td>682</td>
</tr>
</tbody>
</table>

Determine Required Number of Piles
As a starting point, estimate the number of piles needed by calculating the number of piles required to resist the axial load and then add 10 to 20% more piles to resist overturning.

\[
N_{\text{axial}} = \frac{1519}{162} = 9.4 \text{ piles}
\]

Try the trial pile layout presented in Figure 11.4.3.10 with 11 piles.

Knowing the loads applied to the footing and the layout of the piles, the force in each of the pile can be determined. The equation to be used is:

\[
P = \left[ \frac{\text{Axial Load}}{\text{Number of Piles}} \right] + \left[ \frac{M_{\text{par}} \cdot x_{\text{par}}}{\sum x_{\text{par}}^2} \right] + \left[ \frac{M_{\text{perp}} \cdot x_{\text{perp}}}{\sum x_{\text{perp}}^2} \right]
\]

The equation assumes that the footing functions as a rigid plate and that the axial force in the piles due to applied moments is proportional to the distance from the center of the pile group.

\[
\sum x_{\text{par}}^2 = 2 \cdot 3.50^2 + 2 \cdot 1.75^2 + 3 \cdot 0^2 + 2 \cdot (1.75^2) + 2 \cdot (-3.50^2) = 61.25 \text{ ft}^2
\]

\[
\sum x_{\text{perp}}^2 = 3 \cdot 5.00^2 + 2 \cdot 2.50^2 + 1 \cdot 0^2 + 2 \cdot (-2.50^2) + 3 \cdot (-5.00^2) = 175.00 \text{ ft}^2
\]

Then, for example, the Strength I(a) Corner Pile 1 load is:

\[
P = \frac{1531}{11} + 0.350 \cdot 766 \cdot 5.00 = 161.1 \text{ kips}
\]
The factored pile loads at each corner of the footing (as identified in Figure 11.4.3.14) are presented in Table 11.4.3.22. All are below the 162 kip capacity of the piles.

**Table 11.4.3.22 – Factored Pile Loads**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Corner Pile Loads (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Strength I (a)</td>
<td>161.1</td>
</tr>
<tr>
<td>Strength III (c)</td>
<td>71.6</td>
</tr>
<tr>
<td>Strength V(a)</td>
<td>154.4</td>
</tr>
<tr>
<td>Strength V(b)</td>
<td>83.4</td>
</tr>
</tbody>
</table>

**Pile Load Table for Plan**

Piling are driven until dynamic equation measurements indicate the pile has reached refusal or the required design load indicated in the plan. The service load resistance is monitored in the field using the Mn/DOT modified ENR formula given in Section 2452.3E of the Mn/DOT Standard.
Specifications For Construction, 2000 Edition. Designers must calculate the service pile load for the critical load case shown in the plan, using the Standard Plan Note table for piers with piling (see Appendix 2-H).

The critical load case for the pier piling is:
Strength I(a) at Column 2 bottom with $\gamma = 1.25$
Live Load Case 6, and $\Delta$ Temp. = -45° F.

First, compute separate factored pile loads due to dead load, live load, and overturning load for load table:

Factored $P_{DL}$ (includes EV) = $1.25 \cdot (684 + 87.8) + 1.35 \cdot 29.5 = 1004.6$ kips

Factored $M_{LLpar} = 0$ kip – ft

Factored $M_{LLperp} = 0$ kip – ft

Factored $P_U$ (with dynamic load allowance removed)
$= 1.75 \cdot (3.04 - 8.5) = 517.1$ kips

Factored $M_{OTpar} = 0$ kip – ft

Factored $M_{OTperp} = 1.75 \cdot (341) + 169 = 765.8$ kip – ft

Factored Max. Pile Load $P_{DL} = \frac{1004.6}{11 \cdot 2} = 45.7$ tons/pile

Factored $P_{LL} = \frac{517.1}{11 \cdot 2} = 23.5$ tons/pile

Factored $P_{OT} = 0.50 \cdot 18 = 9.0$ kips

Factored $M_{OTpar} = 0$ kip – ft

Factored $M_{OTperp} = 1.75 \cdot (341) + 169 = 765.8$ kip – ft

Factored Max. Pile Load $P_{OT} = \left(\frac{9.0}{11} + \frac{765.8 \cdot 5.00}{175.00} \right) \cdot \frac{1}{2} = 11.3$ tons/pile

Factored Max. Pile Load $P_{total} = 45.7 + 23.5 + 11.3 = 80.5$ tons/pile

Next, compute the maximum service pile load using the loads from Strength I(a) without the load factors. Note that an exception to this is the temperature load. The 0.50 load factor is retained for service because it does not seem reasonable to use a larger load factor for service than that used for strength calculations.

Service $P_{total} = 684 + 87.8 + 29.5 + (304 - 8.5) + (0.50 \cdot 18) = 1105.8$ kips

Service $M_{totalpar} = 0$ kips – ft
Service $M_{\text{total,perp}} = \frac{765.8}{1.75} = 437.6 \text{ kip} - \text{ft}$

Service Max. Pile Load $L_{\text{total}} = \left( \frac{1105.8}{11} + \frac{437.6 \cdot 5.00}{175.00} \right) \cdot \frac{1}{2} = 56.5 \text{ tons/pile}$

Now compute average load factor:
\[
\text{Avg. Load Factor} = \frac{80.5}{56.5} = 1.425
\]

The final results to be shown in the plan are:

<table>
<thead>
<tr>
<th>PIER</th>
<th>Computed Pile Load – Tons/Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PIER</td>
</tr>
<tr>
<td>Factored Dead load</td>
<td>45.7</td>
</tr>
<tr>
<td>Factored Live Load</td>
<td>23.5</td>
</tr>
<tr>
<td>Factored Overturning</td>
<td>11.3</td>
</tr>
<tr>
<td>Factored Total Load</td>
<td>80.5</td>
</tr>
<tr>
<td>* Design Load</td>
<td>56.5</td>
</tr>
</tbody>
</table>

* $80.5 / 1.425 = 56.5 \text{ tons/pile}$

1.425 is Average Load Factor for Strength I Load
Combination

**G. Design Footing**

1. Check Shear Capacity of Footing

Locate Critical Shear and Flexure Sections

Using a column diameter of 3’-0” and a footing thickness of 4’-6”, the critical sections for shear and flexure for the footing can be found. Begin by determining the width of an equivalent square column.

\[
A = \frac{\pi \cdot D^2}{4} = 1018 = b^2 \quad b = 31.9 \text{ in, say 32 in}
\]

The critical section for one-way shear is located a distance $d_v$ away from the face of the equivalent square column. Two-way shear is evaluated on a perimeter located $d_v/2$ away from the face of the actual round column. The same dimension $d_v/2$ is used to check two-way shear for a corner pile.

**[5.8.2.9]**

Estimate $d_v$ as $0.9d_e$. Note that it is not appropriate to use 0.72h here because the tension reinforcement is located so high above the bottom of the footing.
Use an average value for the $d_e$ calculation, assuming #25 bars in both directions and that the bars sit directly on top of the piles.

Then $d_v = 0.9d_e = 0.9 \cdot (54 - 12 - 1.0) = 36.9$ in

**Figure 11.4.3.15**

The critical section for flexure is located at the face of the equivalent square column. All of the critical sections are presented in Figure 11.4.3.15.

**Check One-Way Shear**

The critical one-way shear section is located 36.9 inches away from the face of the equivalent square column.

For the portion of the footing that extends parallel to the pier all of the piles are within the critical shear section and no check is necessary.

For the portion of the footing that extends perpendicular to the pier, the three outermost piles lie outside of the critical shear section and the sum reaction must be resisted.
The one-way shear capacity of the footing is:

\[ \varphi V_c = \varphi \cdot 0.0316 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v \]

\[ \varphi V_c = 0.90 \cdot 0.0316 \cdot 2 \cdot \sqrt{4} \cdot (10 \cdot 12) \cdot 36.9 = 503.7 \text{kips} > 483.9 \text{kips} \quad \text{OK} \]

**Check punching shear around the column**

Assume the entire column vertical load needs to be carried at the perimeter. If the footing has inadequate capacity, reduce the demand by subtracting piles and dead load “inside” of the perimeter.

The perimeter for two-way shear is:

\[ b_o = 2 \cdot \pi \cdot 36.5 = 229.3 \text{ in} \]

The aspect ratio of the column (\( \beta_c \)) is 1.0.

\[ \varphi V_n = \varphi V_c = \varphi \cdot 0.126 \cdot \sqrt{f_c} \cdot b_o \cdot d_v = 0.90 \cdot 0.126 \cdot \sqrt{4} \cdot 229.3 \cdot 36.9 \]

\[ = 1919 \text{kips} > 1531 \text{kips} \quad \text{OK} \]

**Check punching shear on a corner pile**

The critical shear section is assumed to be 0.5\(d_v\) away from the outside edge of the pile. The shear section path with the shortest distance to the edge of the footing will provide the smallest capacity.

\[ b_o = \frac{2 \pi \cdot 24.5}{4} + 18 + 18 = 74.5 \text{ in} \]

Once again using \( \beta_c \) equal to 1.0, inserting values into LRFD Equation 5.13.3.6.3-1 produces:

\[ \varphi V_n = \varphi V_c \cdot \varphi \cdot 0.126 \cdot \sqrt{f_c} \cdot b_o \cdot d_v = 0.90 \cdot 0.126 \cdot \sqrt{4} \cdot 74.5 \cdot 36.9 \]

\[ = 623.5 > 161.1 \text{kips} \quad \text{OK} \]

**2. Design Footing Reinforcement Perpendicular to Pier For Factored Moments**

Determine the required area of flexural reinforcement to satisfy the Strength I(a) Load Combination. Five piles contribute to the design moment at the critical section for moment perpendicular to the pier. The three outer piles are located 44” away from the critical section.

The two inner piles are located 14” away from the critical section.
The design moment on the critical section is:

\[ M_u = \left( 161.1 \cdot \frac{44}{12} \right) + \left( 2 \cdot 150.1 \cdot \frac{14}{12} \right) = 2122 \text{ kip-ft} \]

Set up the equation to solve for the required area of steel:

\[ M_u = 0.90 \cdot A_s \cdot (60) \cdot \left[ d - \frac{A_s \cdot 60}{1.7 \cdot 4 \cdot 120} \right] \cdot \left( \frac{1}{12} \right) \]

\[ 0.3309 \cdot A_s^2 - 4.5 \cdot d \cdot A_s + M_u = 0 \]

\[ A_s = \frac{4.5 \cdot d - \sqrt{20.25 \cdot d^2 - 1.3236 \cdot M_u}}{0.6618} \]

To compute “d” assume that #32 bars are used for both mats of reinforcement and that they rest directly on top of the cut off piles. In addition, reduce “d” to permit either set of bars to rest directly on the pile.

\[ d = \left[ 54 - 12 - 1.27 - \frac{1.27}{2} \right] = 40.10 \text{ in} \]

The required area of steel is 12.02 in². Try 10-#32 bars spaced at 12 inches. The provided area of steel is 12.70 in².

**[5.7.3.4]** Crack Control

Crack control checks are not performed on footings.

**[5.5.3]** Fatigue

By inspection, fatigue is not checked for footings.

**[5.7.3.3.1]** Check Maximum Reinforcement

The depth of the Whitney stress block is:

\[ a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = \frac{12.70 \cdot 60}{0.85 \cdot 4.0 \cdot 120} = 1.87 \text{ in} \]
The depth of the section in compression is:
\[ c = \frac{a}{\beta_1} = \frac{1.87}{0.85} = 2.20 \text{ in} \]

\[ \frac{c}{d} = \frac{2.20}{40.10} = 0.055 < 0.42 \quad \text{OK} \]

**Check Minimum Reinforcement**

The gross moment of inertia is:
\[ I_g = \frac{1}{12} b \cdot t^3 = \frac{1}{12} \cdot 120 \cdot (54)^3 = 1,574,640 \text{ in}^4 \]

The distance from the tension face to the centroid is:
\[ y_t = 27.0 \text{ in} \]

This leads to a cracking moment of:
\[ M_{cr} = \frac{f_r \cdot I_g}{y_t} = \frac{0.48 \cdot 1,574,640}{27.0 \cdot (12)} = 2333 \text{ kip-ft} \]

The minimum required flexural resistance is the lesser of:
\[ 1.2 M_{cr} = 1.2 \cdot 2333 = 2800 \text{ kip-ft} \quad \text{GOVERS} \]
\[ \text{or } 1.33 M_u = 1.33 \cdot 2122 = 2822 \text{ kip-ft} \]

The resisting moment is:
\[ M_r = \phi A_s f_y (d - a/2) = 0.9 \cdot (12.70) \cdot (60) \cdot \left[ 40.10 - \frac{1.87}{2} \right] \cdot \frac{1}{12} \]
\[ = 2238 \text{ kip-ft} < 2800 \text{ kip-ft} \quad \text{No Good} \]

Revise reinforcement to 13-#32 bars spaced at 9 inches \((A_s = 16.51 \text{ in}^2)\) with standard hooks.
\[ M_r = 2889 \text{ kip-ft} > 2800 \text{ kip-ft} \quad \text{OK} \]

**3. Design Footing Reinforcement Parallel to Pier For Factored Moments**

Determine the required area of flexural reinforcement to satisfy the Strength I load combination for parallel moments. Four piles contribute to the design moment at the critical section for moment parallel to the pier.

Piles 1 and 3 have reaction of 161.1 kips and 117.3 kips respectively. The inner pile above the \(X_{par}\) axis was previously shown to have a reaction equal to 150.1 kips.
The pile reaction for the inner pile below the $X_{par}$ axis is:

$$ P = \frac{1531}{11} + \frac{766 \cdot (-2.50)}{175.00} = 128.2 \text{ kips} $$

[5.13.3.6.1] The inner piles lie partially inside of the critical section. Only the portion of the reaction outside the critical section causes moment at the critical section. See Figure 11.4.3.16.

Then the design moment on the critical section is:

$$ M_u = (161.1 + 117.3) \cdot \left( \frac{26}{12} \right) + (150.1 + 128.2) \cdot \left( \frac{11}{12} \right) \cdot \left( \frac{21}{12} \right) = 1050 \text{ kip-ft} $$

Using the same "d" value of 40.10 inches as used for the perpendicular reinforcement, the required area of steel is 5.87 in$^2$. Try 13-#22 bars spaced at 12 inches. The provided area of steel is 7.80 in$^2$.

[5.7.3.4] Crack Control
Crack control checks are not performed on footings.

[5.5.3] Fatigue
By inspection, fatigue is not checked for footings.

[5.7.3.3.1] Check Maximum Reinforcement
The depth of the Whitney stress block is:

$$ a = \frac{A_s \cdot f_y}{0.85 \cdot f'_{c_b}} = \frac{7.80 \cdot 60}{0.85 \cdot 4.0 \cdot 156} = 0.88 \text{ in} $$
The depth of the section in compression is:
\[ c = \frac{a}{\beta_1} = \frac{0.88}{0.85} = 1.04 \text{ in} \]

Compute revised \( d \) for #22 bars:
\[ d = 54 - 12 - 1.27 - \frac{0.875}{2} = 40.29 \text{ in} \]
\[ \frac{c}{d} = \frac{1.04}{40.29} = 0.026 < 0.42 \quad \text{OK} \]

**Check Minimum Reinforcement**

Revise 1.2 \( M_{cr} \) computed earlier for footing length of 13 feet:
\[ 1.2M_{cr} = 2800 \cdot \frac{13}{10} = 3640 \text{ kip} \cdot \text{ft} \]

The minimum required flexural resistance is the lesser of 1.2\( M_{cr} \) or:
\[ 1.33M_u = 1.33 \cdot 1050 = 1397 \text{ kip-ft} \quad \text{GOVERNS} \]

The resisting moment is:
\[ M_r = \phi A_s f_y (d - a/2) = 0.9 \cdot (7.80) \cdot 60 \left[ 40.29 - \frac{0.88}{2} \right] \cdot \frac{1}{12} \]
\[ = 1399 \text{ kip-ft} > 1397 \text{ kip-ft} \quad \text{OK} \]

Provide 13-#22 bars spaced at 12 inches \( (A_s = 7.80 \text{ in}^2) \) with standard hooks.

**4. Dowel Bar Development and Lap Splice**

Determine the lap length for the primary column steel to dowel splice. All primary column steel bars are spliced at the same location, consequently the lap is a Class C splice. The primary column reinforcement consists of #25 epoxy coated bars. For ease of construction, the dowel circle will be detailed to the inside of the column bar circle. Accordingly, the dowels will be increased one size to #29 bars.

\[ \text{[5.7.3.3.2]} \]

The basic development length \( \ell_{db} \) for a #29 bar is the greater of:
\[ \ell_{db} = \frac{1.25 \cdot A_b \cdot f_y}{f_c^{1/4}} = \frac{1.25 \cdot 1.00 \cdot 60}{\sqrt[4]{4}} = 37.50 \text{ in} \quad \text{GOVERNS} \]
or
\[
\ell_{db} = 0.4 \cdot d_b \cdot f_y = 0.4 \cdot 1.128 \cdot 60 = 27.07 \text{ in}
\]

Development length modification factors are:
- 1.5 for epoxy coated bars with concrete cover less than 3 bar diameters.
- 0.8 for bars spaced ≥ 6 inches and side clear cover ≥ 3 inches.
- 0.75 for bars within a spiral with diameter ≥ 0.25 inches and pitch ≤ 4 inches.

The development length \( \ell_d \) of the bars is:
\[
\ell_d = 37.50 \cdot 1.5 \cdot 0.8 \cdot 0.75 = 33.75 \text{ in}
\]

[5.11.5.3.1]

The lap length for a Class C tension splice is:
\[
1.7 \cdot \ell_d = 1.7 \cdot 33.75 = 57.38 \text{ in}
\]

Specify a 4'-10" lap length.

**Dowel Bar Hook Development**
Verify that adequate embedment is provided for the dowel bars in the footing.

The basic development length \( \ell_{hb} \) for a #29 epoxy coated bar with a standard hook is:
\[
\ell_{hb} = \frac{38.0 \cdot d_b}{\sqrt{f'_c}} = \frac{38.0 \cdot 1.128}{\sqrt{4}} = 21.43 \text{ in}
\]

Development length modification factors are:
- 0.7 side cover ≥ 2.5 inches and hook extension cover ≥ 2.0 inches.
- 1.2 for epoxy coated bars.

The development length \( \ell_{dh} \) of the dowel with standard hook is:
\[
\ell_{dh} = 21.43 \cdot 0.7 \cdot 1.2 = 18.00 \text{ in}
\]

The embedment provided is:
\[
\ell_{prov} = 54 - 12 - 1.27 - 0.875 = 39.86 \text{ in} > 18.00 \text{ in} \quad \text{OK}
\]
5. Summary
The footing reinforcement is illustrated in Figure 11.4.3.17.

Figure 11.4.3.17