PURPOSE AND USE OF PROCESS INDUSTRY PRACTICES

In an effort to minimize the cost of process industry facilities, this Practice has been prepared from the technical requirements in the existing standards of major industrial users, contractors, or standards organizations. By harmonizing these technical requirements into a single set of Practices, administrative, application, and engineering costs to both the purchaser and the manufacturer should be reduced. While this Practice is expected to incorporate the majority of requirements of most users, individual applications may involve requirements that will be appended to and take precedence over this Practice. Determinations concerning fitness for purpose and particular matters or application of the Practice to particular project or engineering situations should not be made solely on information contained in these materials. The use of trade names from time to time should not be viewed as an expression of preference but rather recognized as normal usage in the trade. Other brands having the same specifications are equally correct and may be substituted for those named. All Practices or guidelines are intended to be consistent with applicable laws and regulations including OSHA requirements. To the extent these Practices or guidelines should conflict with OSHA or other applicable laws or regulations, such laws or regulations must be followed. Consult an appropriate professional before applying or acting on any material contained in or suggested by the Practice.

This Practice is subject to revision at any time.

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PRINTING HISTORY
March 2005             Issued
July 2007              Technical Correction

Not printed with State funds
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1. Introduction

1.1 Purpose
This Practice establishes guidelines and recommended procedures for use by engineers analyzing and designing heat exchanger and horizontal vessel foundations and should be used where applicable unless otherwise specified.

1.2 Scope
This Practice addresses isolated foundations supported directly on soil. Pile supported footings are not considered in this Practice.

2. References
Applicable requirements of the following Practices and industry codes and standards shall be considered an integral part of this Practice. The edition in effect on the date of contract award shall be used, except as otherwise noted. Short titles will be used herein where appropriate.

2.1 Process Industry Practices (PIP)
– PIP STC01015 - Structural Design Criteria
– PIP STE05121 - Anchor Bolt Design Guide

2.2 Industry Guides and Standards
• American Concrete Institute (ACI)
  – ACI 318/318R-05 - Building Code Requirements for Structural Concrete and Commentary
• American Society of Civil Engineers (ASCE)
• ASTM International (ASTM)
  – ASTM F1554 - Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

3. Definitions

engineer: The engineer who performs the structural design

owner: The party ultimately responsible for contract award. The owner will have authority over the site, facility, structure or project through ownership, lease, or other legal agreement.

stability ratio: The ratio of dead load resisting moment to overturning moment about the edge of rotation

thermal force: The force due to growth between piers caused by a change in temperature of the horizontal vessel or exchanger
4. Design Procedure

4.1 Design Considerations

4.1.1 The engineer should review project design criteria to determine wind and earthquake loadings, corrosion allowances for anchor bolts, anchor bolt types, and any special requirements dictated by the owner.

4.1.2 The engineer should verify that design is based on applicable codes in existence when foundation drawings are issued.

4.2 Vertical Loads

4.2.1 Dead Loads

4.2.1.1 The following nominal loads should be considered as dead loads if applying load factors used in strength design.

a. Structure dead load \( (D_s) \) - Weight of the foundation and soil above the part of the foundation that resists uplift

b. Erection dead load \( (D_f) \) - Fabricated weight of the exchanger or vessel, generally taken from certified exchanger or vessel drawings

c. Empty dead load \( (D_e) \) - Empty weight of the exchanger or vessel including all attachments, trays, internals, bundle, insulation, fireproofing, agitators, piping, ladders, platforms, etc. The eccentric load defined in paragraph 4.2.1.2 should also be added to the empty dead load weight.

d. Operating dead load \( (D_o) \) - Empty dead load of the exchanger or vessel plus the maximum weight of contents (including packing/catalyst) during normal operation. The eccentric load defined in paragraph 4.2.1.2 should also be added to the operating dead load weight.

e. Test dead load \( (D_{t}) \) - (horizontal vessels only) Empty dead load of the vessel plus the weight of test medium contained in the system. The test medium should be as specified in the contract documents or as specified by the owner. Unless otherwise specified, a minimum specific gravity of 1.0 should be used for test medium. Cleaning load should be used for test dead load if cleaning fluid is heavier than test medium. Whether test or cleaning will actually be done in the field should be determined. It is generally desirable to design for test dead load because unforeseen circumstances may occur. The eccentric load defined in paragraph 4.2.1.2 should also be added to the test dead load weight.

4.2.1.2 Eccentric load - Unless more exact information about piping supported on the exchanger or horizontal vessel is available, the following guidelines should be used:
a. A load of an additional 20% of the applicable weight (empty or operating) for exchangers with diameters less than 24 inches

b. A load of an additional 10% of the applicable weight (empty or operating) for exchangers with diameters equal to or greater than 24 inches

c. A load of an additional 10% of the applicable weight (empty, operating, or test) for horizontal vessels

d. This additional load should be applied at a perpendicular horizontal distance of D/2 plus 18 inches from the longitudinal centerline of the vessel, where “D” is the basic diameter (basic diameter = vessel I.D. + 2 times the wall thickness + 2 times the insulation thickness.) This additional eccentric load (vertical load and moment caused by eccentricity) should be distributed to each pedestal in proportion to the distribution of operating load to each pedestal. For stacked exchangers, the weight of only the largest exchanger should be used to estimate the eccentric load.

Comment: These eccentric loads are only guidelines and should be checked against actual conditions when they become available.

4.2.1.3 Load distribution (exchangers) - For most common shell and tube heat exchangers, vertical dead loads should normally be distributed with 60% to the channel end support and 40% to the shell end support. However, the actual exchanger shape and support configuration should be reviewed when determining weight distribution because in many cases load distribution may vary.

4.2.2 Live Loads (L)

4.2.2.1 Live loads should be calculated in accordance with PIP STC01015.

4.2.2.2 Load combinations that include live load in Table 5 and Table 6 in PIP STC01015 do not normally control any portion of the foundation design.

4.3 Horizontal Loads

4.3.1 Wind Loads (W)

4.3.1.1 Wind loads should be calculated in accordance with PIP STC01015.

4.3.1.2 The engineer is responsible for determining wind loads used for the foundation design. Wind loads from vendors or other engineering disciplines should not be accepted without verification.

4.3.1.3 Transverse wind - The wind pressure on the projected area of the side of the exchanger or vessel should be applied as a horizontal shear at the center of the exchanger or vessel. Including the wind loading on projections such as piping, manways, insulation, and platforms during the wind analysis is important. The saddle-to-pier connection should be considered fixed for transverse loads.
4.3.1.4 Longitudinal wind - The wind pressure on the end of the exchanger or vessel should be applied as a horizontal shear at the center of the exchanger or vessel. The flat surface wind pressure on the exposed area of both piers or both columns should also be included, applied as a horizontal shear at the centroid of the exposed area. The saddle-to-pier connection will be considered pinned for longitudinal loads unless more than one row of anchor bolts exists.

4.3.1.5 Shielding - No allowance should be made for shielding from wind by nearby equipment or structures except under unusual conditions.

4.3.2 Earthquake Loads (E)

4.3.2.1 Earthquake loads should be calculated in accordance with PIP STC01015.

4.3.2.2 The engineer is responsible for determining earthquake loads used for the foundation design. Earthquake loads from vendors or other engineering disciplines should not be accepted without verification.

4.3.2.3 For low-friction slide plates (μ ≤ 0.2), all the longitudinal earthquake loads should be applied at the fixed pier. For higher friction slide plates (μ > 0.2), 70% of the earthquake loads should be applied at the fixed pier. Transverse and vertical earthquake loads should be distributed in proportion to the vertical load applied to both piers. The piers are normally designed for the fixed end, and then the pier for the sliding end is made identical, to avoid potential errors in construction and to reduce engineering time. If this proves to be uneconomical, the sliding end should be designed for 30% of the longitudinal earthquake load if using low-friction slide plates, and for 50% of the longitudinal earthquake load if using higher friction slide plates.

4.3.3 Bundle Pull Load (Bp) (Exchangers)

4.3.3.1 Bundle pull load should be calculated in accordance with PIP STC01015.

4.3.3.2 Consideration should be given to reducing the empty weight of the exchanger owing to the removal of the exchanger head (channel) to pull the bundle. The weight of the exchanger head (channel) typically is within the range of 8% to 15% of the empty weight of the exchanger.

4.3.4 Thermal Force

4.3.4.1 Calculate thermal growth using maximum design temperature. Thermal coefficients can be found in Table 1.

4.3.4.2 The thermal force used for design should be the smaller value resulting from the following two calculations:

a. The force required to overcome static friction between the exchanger or vessel support and the slide plate:

\[ F_t = \mu (P_o) \]  

(Equation 1)
where,

\[ F_f = \text{static friction force} \]
\[ \mu = \text{coefficient of friction; refer to the values given in Section 4.6, “Slide Plates”} \]

\[ P_o = \text{nominal operating compression dead load on slide plate} \]

b. The force required to deflect the pier or column an amount equal to half of the thermal growth between exchanger or vessel saddles:

\[ T = \frac{3 \Delta E I}{2 H^3} \quad \text{(Equation 2)} \]

where,

\[ T = \text{force from thermal expansion required to deflect pier or column} \]
\[ \Delta = \text{total growth between exchanger/vessel saddles} = \varepsilon L \]
\[ \varepsilon = \text{thermal expansion coefficient in accordance with Table 1} \]
\[ L = \text{length of exchanger/vessel between saddles} \]
\[ E = \text{modulus of elasticity of concrete pier} \]
\[ I = \text{pier moment of inertia} \]
\[ H = \text{pier height} \]

The thermal force should be applied at the top of the piers.

4.3.5 Load Distribution

The horizontal loads should be divided equally between piers unless otherwise required by Section 4.3 of this Practice.

4.4 Load Combinations

4.4.1 Heat exchangers and horizontal vessel foundations should be designed using load combinations in accordance with PIP STC01015.

4.4.2 Foundations for fin exchangers (double pipe exchangers) should not be designed to resist thermal or bundle pull forces.

4.4.3 Piping thermal loads should be included in combinations when deemed advisable and should be considered as dead loads when applying load factors.

4.5 Anchor Bolts

4.5.1 See PIP STE05121 for anchor bolt design procedures.

4.5.2 Friction force at the bottom of the saddle should be overcome before lateral load is assumed to produce shear in the anchor bolts.
4.5.3 For earthquake loads, horizontal shear forces should be applied to the anchor bolts, assuming no frictional resistance.

4.6 **Slide Plates**

4.6.1 A steel slide plate or low-friction slide plate assembly should typically be provided at the sliding end of every exchanger or vessel regardless of the flexibility inherent in the structural support. Small, lightly loaded exchangers or vessels may not require slide plates.

4.6.2 Low-friction manufactured slide plate assemblies should be used to reduce high-frictional resistance, especially for heavy exchangers or for exchangers with significant thermal growth.

4.6.2.1 For exchangers with bundle pull, steel slide plates instead of low-friction slide plate assemblies may be more cost efficient.

4.6.2.2 Typically, a low-friction slide plate assembly consists of multiple individual slide plate components spaced out along the length of the saddle. Each slide plate component consists of an upper element and a lower element, and the sliding surface is at the interface of the upper and lower elements. The elements should be fabricated with a carbon steel backer plate attached to the elements to facilitate welding of the upper elements to the saddles and the lower elements to the steel bearing plate.

4.6.3 Typical coefficients of friction are as follows. For low-friction slide plate assemblies, manufacturer’s literature should be consulted because coefficients of friction vary with slide plate material, temperature, and pressure.

   a. No slide plate (steel support on concrete) 0.60
   b. Steel slide plate 0.40
   c. Low-friction slide plate assemblies 0.05 to 0.20

4.6.4 Suggested criteria for sizing low-friction slide plate elements are as follows. Manufacturer’s literature should be consulted for temperature restrictions, pressure limitations, and other requirements that may affect the size and types of materials used for the slide plate elements.

Element widths (where $\Delta$ = total thermal growth between exchanger or vessel saddles):

   a. Upper element = saddle width + 1-inch minimum to allow for down-hand welding on the element-to-saddle weld (larger upper element width may be required for exchangers or vessels with large $\Delta$ values).
   
   b. Lower element = upper element width - 2 ($\Delta$) - 1 inch (minimum of 1 inch narrower than upper element)

Element lengths (use 18-inch maximum clear distance between lower elements):

   a. Lower element = based on allowable contact pressure in accordance with the manufacturer’s literature and lower element width
b. Upper element = lower element length + 1 inch

Plates should be aligned with saddle stiffeners where practical.

A continuous steel bearing plate should be provided under the lower elements so that lower elements can be welded to the bearing plate. Minimum width of bearing plate should be 1 inch larger than the width of the lower elements. Minimum length of bearing plate should be 1 inch larger than the saddle length. Bearing stress on concrete should be checked in accordance with ACI 318.

4.6.5 Suggested criteria for sizing steel slide plates are as follows:

a. Minimum width = saddle width + 2 ($\Delta$) + 1 inch

b. Minimum length = saddle length + 1 inch

Bearing stress on concrete should be checked in accordance with ACI 318.

4.7 Pier Design

4.7.1 Pier dimensions should be sized on the basis of standard available forms for the project. When form information is not available, pier dimensions should be sized in 2-inch increments to allow use of standard manufactured forms. Minimum pier dimensions should equal the maximum of the saddle, bearing plate, or steel slide plate dimensions plus 4 inches and should be sized to provide adequate anchor bolt edge distance in accordance with PIP STE05121. Minimum pier width should be no less than 10 inches or 10% of the pier height.

4.7.2 Anchorage Considerations

It is normally desirable to make the pier high enough to contain the anchor bolts and to keep them out of the footing. Consideration must be given to anchor bolt development and foundation depth requirements.

4.7.3 Reinforcement

4.7.3.1 Piers should normally be designed as tension-controlled members (cantilever beams) with two layers of reinforcement. If the pier is a compression-controlled member, the pier should be designed as a column. Size and reinforcement for each pier should normally be the same. Dowel splices are not required if the vertical pier reinforcing projection is less than the larger of 6 ft or the rebar size in feet above the top of footing. For example, #8 rebar can extend up to 8 ft above the mat without dowel splices. For cases that exceed this limit, dowels with minimum projections required for tension splices should be used in accordance with ACI 318.

4.7.3.2 The vertical reinforcement in the piers may need to be increased to account for shear friction. The following formula should be used to calculate the area of reinforcement required for shear friction, $A_{cf}$:

$$A_{cf} = \left[\frac{Vu}{(\mu\phi)} - P_{u\text{pier}}\right]/f_y$$  \hspace{1cm} (Equation 3)

$Vu$ = strength design factored shear force at bottom of pier
\( \mu \) = coefficient of friction, normally use 0.6. If it can be assured that the concrete at the construction joint at the interface between the pedestal and the mat will be intentionally roughened, then 1.0 may be used for \( \mu \).

\( \phi \) = strength reduction factor = 0.75

\( P_{pu} \) = strength design factored axial force at bottom of pier

\( f_y \) = yield strength of vertical reinforcement

4.7.3.3 Minimum reinforcement for piers is #5 at 12 inches on each face with #4 ties at 12 inches. A minimum of two #4 ties (or three ties if moderate or high seismic risk) should be placed within 6 inches of the top of concrete of each pier (not including grout) to protect anchor bolts. All ties should encircle the vertical reinforcement, unless special tie reinforcement for boundary elements is required.

4.7.3.4 For tension-controlled piers, as is normally the case, intermediate ties are not required.

4.8 Column Design

4.8.1 Sizing

Columns (if needed) should be round, square, or rectangular depending on the job criteria or the construction contractor’s preference. Column dimensions should be sized on the basis of standard available forms for the project. If form information is not available, column dimensions should be sized in 2-inch increments to allow use of standard manufactured forms.

4.8.2 Reinforcement

Size and reinforcement for both columns should normally be the same. Dowels should be used to transfer the column loads to the footings. Minimum dowel projections should be as required for a tension splice in accordance with *ACI 318*.

4.9 Footing Design

4.9.1 Sizing

Plan view footing dimensions should be sized on the basis of standard available forms for the project. If form information is not available, footing dimensions should be sized in 2-inch increments to allow use of standard manufactured forms. The footing thickness should be a minimum of 12 inches. Size for both footings should normally be the same. For short exchangers or vessels, a combined footing may be used.

4.9.2 The footing thickness should be adequate for shear and embedment of pier or column reinforcement in accordance with *ACI 318*.

4.9.3 The stability ratio should be in accordance *PIP STC01015*. Stability check is not required for thermal forces.
4.9.4 Soil Bearing

Soil-bearing pressure should be computed for footing design and checked against the allowable pressure using the following formula:

Total footing area in compression \((e \leq b/6)\):

\[
SB = \frac{P}{A} \left[ 1 \pm (6 \frac{e}{b}) \right] \quad \text{Equation 4}
\]

Total footing area not in compression \((e > b/6)\):

\[
SB_{\text{max}} = \frac{2P}{3a (b / 2 - e)} \quad \text{Equation 5}
\]

\[
SB = 0 \text{ at } 3 (b /2 - e) \quad \text{Equation 6}
\]

where,
\(e\) = eccentricity of vertical service load caused by horizontal service load
\(a\) = size of footing perpendicular to direction of horizontal load
\(b\) = size of footing parallel to direction of horizontal load
\(P\) = total vertical service load (exchanger or vessel, pier, footing, and soil)
\(A\) = area of footing

4.9.5 If eccentricity exists in both directions, the equations in paragraph 4.9.4 do not apply. Numerical solutions can be found in many soil mechanics textbooks. Commercial software is also available for such calculations. Figure C is a design aid that provides graphical results based on accurate numerical solutions.

4.9.6 Reinforcement and Stresses

The strength design factored moment and shear should be figured on a unit width strip assuming a simple cantilever. The critical section for moment and diagonal tension shear should be taken at the pier or column face. If shear is excessive, the strength design factored shear should be rechecked using the critical section for shear specified in *ACI 318*. The resulting reinforcing steel should be placed continuously across the entire footing. The minimum amount of bottom reinforcement is \#5 at 12 inches c/c.

4.9.7 Top Reinforcement

Except where seismic effects create tensile stresses, top reinforcement in the footing is not necessary if the factored tensile stress at the upper face of the footing does not exceed the flexural strength of structural plain concrete, as follows:

\[
\gamma_t = 5\phi(\gamma'_c)^{1/2} \quad \text{Equation 7}
\]

where,
\(\gamma_t\) = flexural strength of structural plain concrete, psi
\(\gamma'_c\) = compressive strength of concrete, psi
\( \phi = \) strength reduction factor for structural plain concrete = 0.55

The effective thickness of the footing for tensile stress calculations should be 2 inches less than the actual thickness for footings cast against soil (ACI 318-05, Section R22.7.4). For footings cast against a seal slab, the actual thickness of the footing may be used for the effective thickness. If the factored tensile stress exceeds the flexural strength of structural plain concrete, top reinforcement should be used if an increase in the footing thickness is not feasible.

The following formulas are for calculating the required footing thicknesses with no top reinforcing steel:

For footings cast against soil:

\[ t_{\text{reqd}} = t_{\text{eff}} + 2 \text{ inches} \quad \text{(Equation 8a)} \]

For footings cast against a seal slab:

\[ t_{\text{reqd}} = t_{\text{eff}} \quad \text{(Equation 8b)} \]

With \( t_{\text{eff}} \) calculated as follows:

\[ t_{\text{eff}} = \left( \frac{6M_u}{f'_{t}} \right)^{1/2} \quad \text{(Equation 9)} \]

where,

\( t_{\text{reqd}} = \) required footing thickness with no top reinforcing steel, inches

\( t_{\text{eff}} = \) effective footing thickness, inches

\( M_u = \) strength design factored moment caused by the weight of soil and concrete acting on a 1-inch strip in the footing at the face of the pier, inch-pounds per inch, using a load factor of 1.4

\( f'_{t} = \) flexural strength of structural plain concrete, psi (from Equation 7)

If tensile stress in the upper face of the footing exceeds ACI plain concrete design requirements, top steel should be used if increasing the footing thickness is unfeasible. If top reinforcement is required, minimum reinforcement is #4 at 12 inches c/c.
APPENDIX:

Tables, Figures, and Examples
Table 1 - Thermal Expansion Data

Total linear expansion between 70°F and indicated temperature (inches/100 ft)

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<th>Carbon Steel</th>
<th>Carbon - Moly Low-Chrome (through 3 Cr Mo)</th>
<th>Austenitic Stainless Steels</th>
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Table 1 (continued)

Total linear expansion between 70°F and indicated temperature (inches/100 ft)

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<td>(through 3 Cr Mo)</td>
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</table>
These curves give the approximate weight of standard heat exchangers, all in tons. The curves are for a 192-inch type ET exchanger with two passes in the tubes. The tubes are 3/4 inch on a 90° layout. The tube material is 14-gage steel. For the weights of heat exchangers with other tube lengths, multiply by the following factors:

<table>
<thead>
<tr>
<th>Length in inches:</th>
<th>240</th>
<th>192</th>
<th>168</th>
<th>144</th>
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<td>Heat exchanger factor:</td>
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<td>1.00</td>
<td>0.95</td>
<td>0.90</td>
<td>0.85</td>
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</table>
These curves give the approximate weight of standard tube bundles, all in tons. The tubes are 3/4 inch, 14 gage, and 192 inches long. The tubes are two pass on a square pitch. The baffle spacings range from 8 inches on the 15-inch exchanger to 16 inches on the 48-inch exchanger. For the weight of bundle with other lengths, multiply by the following factors:

- Length in inches: 240 192 168 144 120
- Heat exchanger factor: 1.20 1.00 0.90 0.80 0.70
Figure C - Soil Pressure for Biaxially Loaded Footings

\[ SB_{\text{max}} = K \left( \frac{P}{ab} \right) \]

- Location of \( SB_{\text{max}} \)
- Ratio \( \frac{e_1}{a} \)
- Ratio \( \frac{e_2}{b} \)
- Load \( P \)
Example 1 - Heat Exchanger Foundation

**PLAN**

Dimensions typical both piers

- **A - A**

Top of grout elevation (fixed end)
Top of steel slide plate (sliding end)

- Steel slide plate
  - 3 ft - 1 inch
  - by 11 inches
  - by 3/8 inch

- PIERS
  - PIER (Fixed end)
  - PIER (Sliding end)

- 2 - 1 1/4 inch diameter
  - ASTM F1554, Grade 36
  - anchor bolts per pier
  - P = 4 inches (fixed end w/1 nut)
  - P = 5 1/4 inches (sliding end w/2 nuts)

- # 4 ties @ 11 inches
- *# 6 @ 10 inches each way*
- *# 4 @ 10 inches each way*
(Example 1, continued)

**DESIGN DATA**

**Exchanger Data:**

Empty weight = 32 kips each  
Operating weight = 44 kips each  
Bundle weight = 19 kips each  
Channel weight = 3.5 kips each  
Basic diameter = 42 inches, or 3.5 ft  
Max. design temperature = 550°F  
Exchanger material: carbon steel  
Bolts: 2 - 1-1/4-inch diameter  
\textit{ASTM F1554}, Grade 36 (galvanized) per pier  
Bolt spacing = 2 ft - 8 inch c/c  
Saddle: 3 ft - 0 inch by 9 inches  
Load distribution: 60% at channel end, 40% at shell end

**Design Criteria:**

Concrete: $f'_c = 4,000$ psi  
Reinforcing: $f_y = 60,000$ psi  
Soil unit weight: $\gamma = 100$ pcf  
Allowable net soil bearing: $SB_{net} = 5.5$ ksf (at 4-ft depth)  
Wind load: \textit{ASCE/SEI 7-05}  
Earthquake load: \textit{ASCE/SEI 7-05}

**DETERMINE LOADS**

**Empty and Operating Loads**

Exchanger weight supplied by outside manufacturers does not include the weight of attached pipes and insulation. Increase exchanger weight by 10% of the larger exchanger to account for these attached items (refer to this Practice, Section 4.2, vertical loads, empty and operating dead loads).

\[
\begin{align*}
\text{Empty dead load} (D_e) &= 32\text{ kips} + (32\text{ kips})(1.1) = 67.2\text{ kips} \\
\text{Operating dead load} (D_o) &= 44\text{ kips} + (44\text{ kips})(1.1) = 92.4\text{ kips}
\end{align*}
\]

<table>
<thead>
<tr>
<th></th>
<th>40% at Shell/Fixed End</th>
<th>60% at Channel/Sliding End</th>
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</thead>
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<tr>
<td>Empty dead load</td>
<td>26.9 kips</td>
<td>40.3 kips</td>
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<tr>
<td>Operating dead</td>
<td>37.0 kips</td>
<td>55.4 kips</td>
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<tr>
<td>load (D_o)</td>
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</table>

**Transverse Moment from Pipe Eccentricity**

Eccentricity = (basic diameter)/2 + (1.5 ft) = (3.5 ft)/2 + (1.5 ft) = 3.25 ft  
Empty $M_{Te}$ (channel end) = (32 kips)(0.1)(0.6 channel end)(3.25 ft) = 6.24 ft-k  
Empty $M_{Te}$ (shell end) = (32 kips)(0.1)(0.4 shell end)(3.25 ft) = 4.16 ft-k
Operating \( M_{Te} \) (channel end) = (44 kips)(0.1)(0.6 channel end)(3.25 ft) = 8.58 ft-k
Operating \( M_{Te} \) (shell end) = (44 kips)(0.1)(0.4 shell end)(3.25 ft) = 5.72 ft-k

**Wind Loads**

Wind load calculations are beyond the scope of this Practice.
Exchanger wind load is applied at the center of each exchanger.
Transverse wind: \( H_w = 1.28 \) kips (per exchanger)
Longitudinal wind: \( H_w = 0.25 \) kips (per exchanger)
Transverse or longitudinal wind on each pier: \( H_w = 0.039 \) ksf

**Earthquake Loads**

Earthquake load calculations are beyond the scope of this Practice.
Exchanger earthquake loads are applied at the center of each exchanger.
Note that the following are strength design loads:

<table>
<thead>
<tr>
<th></th>
<th>Empty (Per Exchanger)</th>
<th>Operating (Per Exchanger)</th>
<th>Pier</th>
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</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>5.42 kips</td>
<td>7.45 kips</td>
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<tr>
<td>Longitudinal</td>
<td>8.80 kips</td>
<td>12.10 kips</td>
<td>0.250 W</td>
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</table>

For calculations based on allowable stress design (service loads), the strength design loads shown in the preceding table should be converted to service loads by multiplying by 0.7, in accordance with *PIP STC01015*, Table 5.

<table>
<thead>
<tr>
<th></th>
<th>Empty (Per Exchanger)</th>
<th>Operating (Per Exchanger)</th>
<th>Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>3.79 kips</td>
<td>5.22 kips</td>
<td>0.108 W</td>
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<tr>
<td>Longitudinal</td>
<td>6.16 kips</td>
<td>8.47 kips</td>
<td>0.175 W</td>
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</tbody>
</table>

**Bundle Pull**

\( V_{Bp} = 1.0 \) (bundle weight) = 1.0 (19 k) = 19.0 kips

The minimum is the lesser of 2 kips or exchanger weight (*PIP STC01015*, Section 4.1.8). Therefore, use the bundle weight.

- Use \( V_{Bp} = 19.0 \) kips (applied at centerline of top exchanger.)

Note that a reduction in the empty load of the exchanger owing to the removal of the exchanger head (channel) to pull the bundle is not included in this foundation calculation because the reduction in the empty load is not considered to have a significant effect on the design.

**Thermal Force**

1. Compute sliding force (assume that a steel slide plate is used):

   Coefficient of friction, \( \mu = 0.40 \) (this Practice, Section 4.6.3)

   Operating load,
2. Compute force required to deflect pier:

Assume pier is 42 inches long by 16 inches wide by 78 inches high.

Moment of inertia,

\[ I = bh^3 / 12 = (42 \text{ inches})(16 \text{ inches})^3 / 12 = 14,336 \text{ inches}^4 \]

Modulus of elasticity, \( Ec = 57,000 \frac{f'_c}{4,000} = 3,605 \text{ ksi} \) (ACI 318-05, Section 8.5.1)

Thermal expansion coefficient for carbon steel at 550°F: \( \varepsilon = 0.0411 \text{ inch/ft} \) (this Practice, Table 1)

Thermal growth between saddles,

\[ \Delta = (\varepsilon)(L) = (0.0411 \text{ inch/ft})(11 \text{ ft}) = 0.452 \text{ inches} \]

\[ T = 3 \Delta E I / 2 H^3 = 3 (0.452 \text{ inch})(3,605 \text{ ksi})(14,336 \text{ inches}^4) / 2 (78 \text{ inches})^3 = 73.8 \text{ kips} \] (this Practice, Equation 2)

- Because \( F_f < T \) and because a lower friction factor will not help the distribution of earthquake and bundle pull loads, use steel slide plate.

### DESIGN ELEMENTS

#### Size Steel Slide Plate

Width = (saddle width) + 2(\Delta) + 1 inch = (9 inches) + 2 (0.452 inches) + (1 inch) = 10.90 inches, say 11 inches

Length = (saddle length) + 1 inch = (36 inches) + (1 inch) = 3 ft - 1 inch

Check bearing stress (operating and longitudinal earthquake):

\[ P_{Eo} = (12.10 \text{ kips})(2.75 \text{ ft} + 8.25 \text{ ft}) / (11 \text{ ft between piers}) = 12.1 \text{ kips} \] (downward load caused by overturning)

\[ P_u = 1.2 (P_o) + 1.0 (P_{Eo}) = 1.2 (55.4 \text{ kips}) + 1.0 (12.1 \text{ kips}) = 78.6 \text{ kips} \] (PIP STC01015, Table 6, Load Combination 3)

\[ P_n = \phi 0.85 f'_c A_1 = (0.65)(0.85)(4 \text{ ksi})(11 \text{ inches})(37 \text{ inches}) = 899 \text{ kips} > P_u \] (ACI 318-05, Section 10.17)

- Use a steel slide plate that is 3 ft - 1 inch by 11 inches by 3/8 inches.

#### Pier Size

Pier length

\( (c/c \text{ bolts}) + (2)(5\text{-inch minimum anchor bolt edge distance}) = (2 \text{ ft - 8 inches}) + 2 (5 \text{ inches}) \)

= 3 ft - 6 inches \( \leftarrow \) controls

\( (\text{steel slide plate length}) + (4 \text{ inches}) = (3 \text{ ft - 1 inch}) + (4 \text{ inches}) \)
 = 3 ft - 5 inches

Pier width

10 inches

10% of pier height = (0.10)(78 inches) = 7.8 inches (based on assumed pier height)

(2)(5-inch minimum anchor bolt edge distance) = 2 (5 inches) = 10 inches

(steel slide plate width) + (4 inches) = (11 inches) + (4 inches) = 15 inches ← controls,
but use 16 inches for forming in 2-inch increments

• Use a pier size of 1 ft - 4 inches by 3 ft - 6 inches.

Anchor Bolt Design

Anchor bolt design is beyond the scope of this Practice. Refer to PIP STE05121 for procedures.

• Use two 1-1/4-inch diameter, ASTM F1554, Grade 36 anchor bolts per pier.

Pier Design

At base of pier (assume footing to be 1.5 ft thick):

Pier height = 8.0 ft - 1.5 ft = 6.5 ft

Pier weight = (0.15 kcf)(1.33 ft)(3.5 ft)(6.5 ft) = 4.54 kips

Use load combinations and strength design load factors from PIP STC01015, Table 6.

Operating and longitudinal earthquake at fixed end:

Apply 70% of exchanger earthquake loads at fixed end (this Practice, Paragraph 4.3.2.3,)

Horizontal load at fixed end,

\[ V_{uFX} = 1.0 \left[ (0.7)(12.10 \text{kips})(2 \text{ exchangers}) + (0.25)(4.54 \text{kips}) \right] = 16.94 \text{kips} + 1.14 \text{kips} \]

\[ = 18.08 \text{kips} \quad (PIP \ STC01015, \ Table \ 6, \ Load \ Combination \ 3) \]

Shear and moment at bottom of pier,

\[ V_{uFX} = 16.94 \text{kips} + 1.14 \text{kips} = 18.08 \text{kips} \]

\[ M_{uFX} = (16.94 \text{kips})(6.5 \text{ ft}) + (1.14 \text{kips})(6.5 \text{ ft}/2) = 113.8\text{ft-k} \]

Empty and bundle pull at fixed end:

Bundle pull force,

\[ V_{ulp} = 1.6(19.0 \text{kips}) = 30.4 \text{kips} \quad (PIP \ STC01015, \ Table \ 6, \ Load \ Combination \ 9 \ or \ 10) \]

Vertical load at top of pier due to bundle pull on top exchanger,

\[ P_{ulp} = (30.4 \text{kips})(2.75 \text{ ft} + 5.5 \text{ ft}) / (11 \text{ ft between piers}) = 22.8 \text{kips} \]

Net vertical load on sliding pier pushing top bundle in (use 0.9D for load factor, PIP STC01015, Table 6, Load Combination 10),

\[ P_{uSL} = (0.9)(40.3 \text{kips}) - (22.8 \text{kips}) = 13.5 \text{kips} \]

Horizontal load at sliding end,

\[ V_{uSL} = \mu (P_{uSL}) = 0.40 (13.5 \text{kips}) = 5.40 \text{kips} < 1/2 \text{ bundle pull force} \ (V_{ulp}) \]
Horizontal load at fixed end,

\[ V_{uFX} = (V_{uBP}) - (V_{uSL}) = (30.4 \text{ kips}) - (5.40 \text{ kips}) = 25.0 \text{ kips} \]

Shear and moment at bottom of pier,

\[ V_{ufX} = 25.0 \text{ kips} \]
\[ M_{ufX} = (25.0 \text{ kips})(6.5 \text{ ft}) = 162.5 \text{ ft-k} \]

Operating and thermal at fixed end:

Thermal force,

\[ V_{uThermal} = 1.4(22.2 \text{ kips}) = 31.08 \text{ kips} \]

Horizontal load at fixed end,

\[ V_{uFX} = V_{uThermal} = 31.08 \text{ kips} \]

Shear and moment at bottom of pier,

\[ V_{ufX} = 31.08 \text{ kips} \]
\[ M_{ufX} = (31.08 \text{ kips})(6.5 \text{ ft}) = 202.0 \text{ ft-k} \]

Check diagonal tension shear:

\[ d = (16\text{-inch pier}) - (2\text{-inch clear}) - (0.5\text{-inch ties}) - (\text{say } 1.0\text{-inch bar}) / 2 = 13.0 \text{ inches} \]

\[ \phi \ V_c = \phi \ 2 \ \sqrt{f'c} \ b_w \ d \] \hspace{1cm} (ACI 318-05, Equation 11-3)

\[ = (0.75)(2) \sqrt{4,000 \text{ psi}} \ (42 \text{ inches})(13.0 \text{ inches}) / 1,000 = 51.8 \text{ kips} > V_u = 31.08 \text{ kips} \hspace{0.5cm} \text{OK} \]

\[ 0.5 \ \phi \ V_c = (0.5)(51.8 \text{ kips}) = 25.9 \text{ kips} < V_u = 31.08 \text{ kips} \]

Minimum tie requirements from Section 4.7.3.3 of this Practice is #4 ties at 12-inch spacing; however, because \( V_u > 0.5 \phi \ V_c \), spacing requirement should be checked for #4 ties to meet minimum shear reinforcement requirements of ACI 318-05, Section 11.5.5:

\[ A_v = 0.75 \ \sqrt{f'c} \ b_w \ s / f_y \] \hspace{0.5cm} but not less than \( 50 \ b_w \ s / f_y \)

\[ s_{req'd} = A_v \ f_y / 0.75 \ \sqrt{f'c} \ b_w = (0.20 \text{ in}^2)(2)(60,000 \text{ psi}) / (0.75)\sqrt{4000 \text{ psi}} \ (42 \text{ inches}) \]
\[ = 12.0 \text{ inches but not more than } A_v \ f_y / 50 \ b_w \]
\[ = (0.20 \text{ in}^2)(2)(60,000 \text{ psi}) / (50)(42 \text{ inches}) \]
\[ = 11.4 \text{ inches} \hspace{0.5cm} \text{controls} \]

- Use #4 ties at 11-inch spacing.

Design for moment:

\[ F = b \ d^2 / 12,000 = (42 \text{ inches})(13.0 \text{ inches})^2 / 12,000 = 0.592 \]
\[ K_u = M_u / F = (202.0 \text{ ft-k}) / (0.592) = 341.2 \rightarrow \rho = 0.00674 \]
\[ A_s = \rho \ b \ d = (0.00674)(42 \text{ inches})(13.0 \text{ inches}) = 3.68 \text{ inches}^2 \]

The following equation is provided for illustration only; it should not control unless \( f'c > 4,440 \text{ psi} \).
\[ A_{s_{\text{min}}} = 3 \frac{\sqrt{f'_{c}}}{b_w d} / f_y = 3 \frac{\sqrt{4,000 \text{ psi}}}{42 \text{ inches}}(13.0 \text{ inches}) / (60,000 \text{ psi}) \]
\[ = 1.73 \text{ inches}^2 \quad (ACI 318-05, \text{Equation 10-3}) \]
\[ A_{s_{\text{min}}} = 200 \frac{b_w d}{f_y} = 200 (42 \text{ inches})(13.0 \text{ inches}) / (60,000 \text{ psi}) = 1.82 \text{ inches}^2 \]
\[ (ACI 318-05, \text{Section 10.5.1}) \]

Find total \( A_s \) requirement including shear friction, \( A_{vfr} \) at fixed end with \( LF = 1.4 \) for \((P_o + \text{pier weight}) \) at bottom of pier:
\[ A_{vfr} = \frac{V_u/\mu}{{\phi}} - P_{u_{\text{pier}}} = \frac{(31.08 \text{ k})/(0.6)(0.75) - (1.4)(37.0 \text{ k} + 4.54 \text{ k})}{60 \text{ ksi}} \]
\[ = [69.07 - 58.16] / 60 = 0.18 \text{ inches}^2 \quad (\text{this Practice, Equation 3}) \]
\[ A_s (\text{total on each face}) = A_s (\text{moment}) + A_{vfr}/2 = 3.68 + 0.18/2 = 3.77 \text{ inches}^2 \leftarrow \text{controls} \]

- Use five #8 bars each face (\( A_s \) provided = 3.95 inches\(^2\)).

Determine #8 splice length:
\[ l_d / d_b = \frac{3 f_y (\Psi)(\Psi)(\Psi)(\lambda)}{40 \sqrt{f'_{c}}(c + K_n / d_b)} = \frac{3(60,000 \text{ psi})(1.0)(1.0)(1.0)}{40 \sqrt{4,000 \text{ psi}(2.5)}} = 28.5 \]
\[ (ACI 318-05, \text{Section 12.2.3}) \]
\[ l_d = (28.5) d_b = (28.5)(1.0 \text{ inch}) = 28.5 \text{ inches} \]
Class B splice = 1.3 \( (l_d) = 1.3 \( (28.5 \text{ inches}) = 37.1 \text{ inches} \quad (ACI 318-05, \text{Section 12.15.1}) \]
- Do not use a splice because the pier height is 6 ft - 6 inches < 8 ft -0 inch for #8 bar.
- (this Practice, Section 4.7.3.1)

**Footing Size**

Determine minimum footing thickness to develop standard hook for #8 pier reinforcing:
\[ l_{dh} = (0.02\Psi e f_y / \sqrt{f'_{c}})(d_h)(0.7)(A_{s_{\text{required}}}/A_{s_{\text{provided}}}) \]
\[ = [(0.02)(1.0)(1.0)(60,000 \text{ psi}) / \sqrt{4,000 \text{ psi}} ](1 \text{ inch})(0.7)(3.77 \text{ in}^2/3.95 \text{ in}^2) = 12.7 \text{ inches} \]
Minimum thickness = (12.7 inches) + (2 layers)(0.75-inch rebar) + (3 inches clear) = 17.2 inches
- Use 18-inch footing thickness.

\( SB_{\text{allow}} = (5.5 \text{ ksf net}) + (4.0 \text{ ft deep})(0.10 \text{ kcf soil}) = 5.9 \text{ ksf gross} \)
- Try an 8-ft by 5.5-ft footing, 1.5 ft thick.

Weights:
- Pier = (0.15 kcf)(3.50 ft)(1.33 ft)(8 ft - 1.5 ft) = 4.54 kips
- Footing = (0.15 kcf)(8 ft)(5.5 ft)(1.5 ft) = 9.90 kips
- Soil = (0.10 kcf) [(8 ft)(5.5 ft) - (3.50 ft)(1.33 ft)] (4 ft - 1.5 ft) = 9.84 kips
- Total = (4.54 k) + (9.90 k) + (9.84 k) = 24.28 kips

Soil-bearing and stability ratio checks:
Use load combinations for allowable stress design (service loads) from \( PIP STC01015, \text{Table 5} \)
Check operating and thermal and eccentric (channel/sliding end):

Thermal force at top of pier,

\[ V_{\text{Thermal}} = 22.2 \text{ kips} \]

Maximum axial load at bottom of footing,

\[ P_{\text{max}} = P_s + P_o = (24.28 \text{ k}) + (55.4 \text{ k}) = 79.68 \text{ kips} \]

\[(PIP STC01015, \text{ Table 5, Load Combination 1})\]

Moments at bottom of footing,

\[ M_L = (22.2 \text{ k})(8 \text{ ft}) = 177.6 \text{ ft-kips} \]
\[ M_{To} \text{ (from pipe eccentricity)} = 8.58 \text{ ft-kips} \]

Check soil bearing using maximum axial load,

\[ e_1 = \frac{M_L}{P_{\text{max}}} = \frac{(177.6 \text{ ft-k})}{(79.68 \text{ k})} = 2.23 \text{ ft} \]
\[ e_2 = \frac{M_{To}}{P_{\text{max}}} = \frac{(8.58 \text{ ft-k})}{(79.68 \text{ k})} = 0.108 \text{ ft} \]

\[ e_1 / a = \frac{(2.23 \text{ ft})}{(8 \text{ ft})} = 0.279 \]
\[ e_2 / b = \frac{(0.108 \text{ ft})}{(5.5 \text{ ft})} = 0.020 \]

Read Figure C, this Practice: \[ K = 3.20 \]

\[ SB_{\text{max}} = K \left( \frac{P_{\text{max}}}{ab} \right) = (3.20)\left(\frac{(79.68 \text{ k})(8 \text{ ft})(5.5 \text{ ft})}{(88.15 \text{ k})(8 \text{ ft})(5.5 \text{ ft})}\right) = 5.79 \text{ ksf} < SB_{\text{allow}} = 5.9 \text{ ksf} \quad \text{OK} \]

Check operating and longitudinal earthquake and eccentric (channel/sliding end):

Longitudinal operating earthquake load on exchangers,

\[ 0.7 V_{\text{LEo}} = 8.47 \text{ kips} \]

Vertical load at top of piers from longitudinal operating earthquake load on exchangers (owing to overturning moment),

\[ 0.7 P_{\text{Eo}} = (8.47 \text{ kips})(2.75 \text{ ft} + 8.25 \text{ ft}) / (11.0 \text{ ft}) = \pm 8.47 \text{ kips} \]

Axial loads at bottom of footing,

\[ P_{\text{max}} = P_s + P_o + 0.7 P_{\text{Eo}} = (24.28 \text{ k}) + (55.4 \text{ k}) + (8.47 \text{ k}) = 88.15 \text{ kips} \]

\[(PIP STC01015, \text{ Table 5, Load Combination 3})\]

\[ P_{\text{min}} = 0.9 (P_s + P_o) - 0.7 P_{\text{Eo}} = (0.9)(24.28 \text{ k} + 55.4 \text{ k}) - (8.47 \text{ k}) = 63.24 \text{ kips} \]

\[(PIP STC01015, \text{ Table 5, Load Combination 5a})\]

Moments at bottom of footing,

\[ M_L = (0.3 \text{ at sliding end})(8.47 \text{ k})(2 \text{ exchangers})(8.0 \text{ ft}) \]
\[ + (0.175)(4.54 \text{ k pier wt})(6.5 \text{ ft} / 2 + 1.5 \text{ ft}) \]
\[ = 44.43 \text{ ft-kips} \]
\[ M_{To} \text{ (from pipe eccentricity)} = 8.58 \text{ ft-kips} \]

Soil-bearing check using maximum axial load,

\[ e_1 = \frac{M_L}{P_{\text{max}}} = \frac{(44.43 \text{ ft-k})}{(88.15 \text{ k})} = 0.504 \text{ ft} \]
\[ e_2 = \frac{M_{To}}{P_{\text{max}}} = \frac{(8.58 \text{ ft-k})}{(88.15 \text{ k})} = 0.097 \text{ ft} \]
\[ e_1 / a = (0.504 \text{ ft}) / (8 \text{ ft}) = 0.063 \quad e_2 / b = (0.097 \text{ ft}) / (5.5 \text{ ft}) = 0.018 \]

Read Figure C, this Practice: \( K = 1.50 \)

\[ SB_{max} = K \left( \frac{P_{max}}{ab} \right) = (1.50) \left[ \frac{(88.15 \text{ k})}{(8 \text{ ft})(5.5 \text{ ft})} \right] = 3.00 \text{ ksf} < SB_{allow} = 5.9 \text{ ksf} \quad \text{OK} \]

Stability ratio check using minimum axial load,

\[ OTM_L (\text{overturning moment}) = [M_L + (0.7P_{Eo})(a/2)] = [(44.43 \text{ ft-k}) + (8.47 \text{ k})(8 \text{ ft})/2] \]
\[ = 78.31 \text{ ft-k} \]

\[ RM_L (\text{resisting moment}) = 0.9(P_s + P_o)(a/2) = (0.9)(24.28 \text{ k} + 55.4 \text{ k})(8 \text{ ft})/2 = 286.8 \text{ ft-k} \]

\[ \text{Stability ratio} = \frac{RM_L}{OTM_L} = \frac{286.8 \text{ ft-k}}{(78.31 \text{ ft-k})} = 3.66 > 1.0 \]

Check operating and longitudinal earthquake and eccentric (shell/fixed end):

Longitudinal operating earthquake load on exchangers,

\[ 0.7 \ V_{L E_o} = 8.47 \text{ kips} \text{ applied at the center of each exchanger} \]

Vertical load at top of piers from longitudinal operating earthquake load on exchangers (owing to overturning moment),

\[ 0.7 \ P_{Eo} = (8.47 \text{ kips})(2.75 \text{ ft} + 8.25 \text{ ft}) / (11.0 \text{ ft}) = \pm 8.47 \text{ kips} \]

Axial loads at bottom of footing,

\[ P_{max} = P_s + P_o + 0.7 \ P_{Eo} = (24.28 \text{ k}) + (37.0 \text{ k}) + (8.47 \text{ k}) = 69.75 \text{ kips} \]

\((PIP \ STC01015, \text{ Table 5, Load Combination 3})\)

\[ P_{min} = 0.9 \ (P_s + P_o) - 0.7 \ P_{Eo} = (0.9)(24.28 \text{ k} + 37.0 \text{ k}) - (8.47 \text{ k}) = 46.68 \text{ kips} \]

\((PIP \ STC01015, \text{ Table 5, Load Combination 5a})\)

Moments at bottom of footing,

\[ M_L = (0.7 \text{ at fixed end})(8.47 \text{ k})(2 \text{ exchangers})(8.0 \text{ ft}) \]
\[ + (0.175)(4.54 \text{ k pier wt})(6.5 \text{ ft}/2 + 1.5 \text{ ft}) \]
\[ = 98.64 \text{ ft-kips} \]

\[ M_{To} (\text{from pipe eccentricity}) = 5.72 \text{ ft-kips} \]

Soil-bearing check using maximum axial load,

\[ e_1 = M_L / P_{max} = (98.64 \text{ ft-k}) / (69.75 \text{ k}) = 1.41 \text{ ft} \]

\[ e_2 = M_{To} / P_{max} = (5.72 \text{ ft-k}) / (69.75 \text{ k}) = 0.082 \text{ ft} \]

\[ e_1 / a = (1.41 \text{ ft}) / (8 \text{ ft}) = 0.176 \quad e_2 / b = (0.082 \text{ ft}) / (5.5 \text{ ft}) = 0.015 \]

Read Figure C, this Practice: \( K = 2.15 \)

\[ SB_{max} = K \left( \frac{P_{max}}{ab} \right) = (2.15) \left[ \frac{(69.75 \text{ k})}{(8 \text{ ft})(5.5 \text{ ft})} \right] = 3.41 \text{ ksf} < SB_{allow} = 5.9 \text{ ksf} \quad \text{OK} \]

Stability ratio check using minimum axial load,

\[ OTM_L (\text{overturning moment}) = [M_L + (0.7P_{Eo})(a/2)] = [(98.64 \text{ ft-k}) + (8.47 \text{ k})(8 \text{ ft})/2] \]
\[ = 132.5 \text{ ft-k} \]

\[ RM_L (\text{resisting moment}) = 0.9(P_s + P_o)(a/2) = (0.9)(24.28 \text{ k} + 37.0 \text{ k})(8 \text{ ft})/2 = 220.6 \text{ ft-k} \]
Stability ratio = \( \frac{R_{ML}}{OT_{ML}} = \frac{(220.6 \text{ ft-k})}{(132.5 \text{ ft-k})} = 1.66 > 1.0 \quad \text{OK} \)

Check empty and longitudinal earthquake and eccentric (channel/sliding end) loads:

Longitudinal empty earthquake load on exchangers,

\[ 0.7 \, V_{LEE} = 6.16 \text{ kips} \]

Vertical load at top of piers from longitudinal empty earthquake load on exchangers (owing to overturning moment),

\[ 0.7 \, P_{Ec} = \frac{(6.16 \text{ kips})(2.75 \text{ ft} + 8.25 \text{ ft})}{(11.0 \text{ ft})} = \pm 6.16 \text{ kips} \]

Minimum axial load at bottom of footing,

\[ P_{min} = 0.9 \,(P_s + P_e) - 0.7 \, P_{Ec} = (0.9)(24.28 \text{ k} + 40.3 \text{ k}) - (6.16 \text{ k}) = 51.96 \text{ kips} \]

\( (PIP \ STC01015, \ Table \ 5, \ Load \ Combination \ 5b) \)

Longitudinal moment at bottom of footing,

\[ M_L = (0.3 \text{ at sliding end})(6.16 \text{ k})(2 \text{ exchangers})(8.0 \text{ ft}) \]
\[ + (0.175)(4.54 \text{ k pier wt})(6.5 \text{ ft/2} + 1.5 \text{ ft}) \]
\[ = 33.34 \text{ ft-kips} \]

Stability ratio check using minimum axial load,

\[ OT_{ML} \ (overturning \ moment) = [M_L + (0.7P_{Ec})(a/2)] = [(33.34 \text{ ft-k}) + (6.16 \text{ k})(8 \text{ ft})/2] = 57.98 \text{ ft-k} \]

\[ R_{ML} \ (resisting \ moment) = 0.9(P_s + P_e)(a/2) = (0.9)(24.28 \text{ k} + 40.3 \text{ k})(8 \text{ ft})/2 = 232.5 \text{ ft-k} \]

Stability ratio = \( \frac{R_{ML}}{OT_{ML}} = \frac{(232.5 \text{ ft-k})}{(57.98 \text{ ft-k})} = 4.01 > 1.0 \)

Check empty and longitudinal earthquake and eccentric (shell/fixed end):

Longitudinal operating earthquake load on exchangers,

\[ 0.7 \, V_{LEE} = 6.16 \text{ kips} \]

Vertical load at top of piers from longitudinal empty earthquake load on exchangers (owing to overturning moment),

\[ 0.7 \, P_{Ec} = \frac{(6.16 \text{ kips})(2.75 \text{ ft} + 8.25 \text{ ft})}{(11.0 \text{ ft})} = \pm 6.16 \text{ kips} \]

Minimum axial load at bottom of footing,

\[ P_{min} = 0.9 \,(P_s + P_e) - 0.7 \, P_{Ec} = (0.9)(24.28 \text{ k} + 26.9 \text{ k}) - (6.16 \text{ k}) = 39.90 \text{ kips} \]

\( (PIP \ STC01015, \ Table \ 5, \ Load \ Combination \ 5b) \)

Longitudinal moment at bottom of footing,

\[ M_L = (0.7 \text{ at fixed end})(6.16 \text{ k})(2 \text{ exchangers})(8.0 \text{ ft}) \]
\[ + (0.175)(4.54 \text{ k pier wt})(6.5 \text{ ft/2} + 1.5 \text{ ft}) \]
\[ = 72.77 \text{ ft-kips} \]

Stability ratio check using minimum axial load,

\[ OT_{ML} \ (overturning \ moment) = [M_L + (0.7P_{Ec})(a/2)] = [(72.77 \text{ ft-k}) + (6.16 \text{ k})(8 \text{ ft})/2] \]
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= 97.41 ft-k

\[ \text{RM}_L \text{ (resisting moment)} = 0.9(P_s + P_o)(a/2) = (0.9)(24.28 \text{ k} + 26.9 \text{ k})(8 \text{ ft})/2 = 184.2 \text{ ft-k} \]

Stability ratio = \( \text{RM}_L / \text{OTM}_L = (184.2 \text{ ft-k}) / (97.41 \text{ ft-k}) = 1.89 > 1.0 \)

Check operating and transverse earthquake and eccentric (channel/sliding end):

Transverse operating earthquake load on exchangers,

0.7 \( V_{TEo} \) = 5.22 kips applied at the center of each exchanger

Axial loads at bottom of footing,

\[ \text{P}_{\text{max}} = P_s + P_o = (24.28 \text{ k}) + (55.4 \text{ k}) = 79.68 \text{ kips} \]

\[ (PIP STC01015, \text{ Table 5, Load Combination 3}) \]

\[ \text{P}_{\text{min}} = 0.9 \ (P_s + P_o) = (0.9)(24.28 \text{ k} + 55.4 \text{ k}) = 71.71 \text{ kips} \]

\[ (PIP STC01015, \text{ Table 5, Load Combination 5a}) \]

Transverse moment at bottom of footing,

\[ M_T = (5.22 \text{ kips}) \ [(2.75 \text{ ft} + 8.0 \text{ ft}) + (5.5 \text{ ft} + 2.75 \text{ ft} + 8.0 \text{ ft})] \ (0.6 \text{ channel end}) \]

\[ + (0.108)(4.54 \text{ k pier wt})(6.5 \text{ ft/2} + 1.5 \text{ ft}) + (8.58 \text{ ft-k pipe eccentricity}) \]

\[ = 95.47 \text{ ft-kips} \]

Soil-bearing check using maximum axial load,

\[ e = M_T / \text{P}_{\text{max}} = (95.47 \text{ ft-k}) / (79.68 \text{ k}) = 1.20 \text{ ft} > b/6 = (5.5 \text{ ft})/6 = 0.92 \]

\[ \text{SB}_{\text{max}} = 2 \text{P}_{\text{max}} / [3 \text{ a} (b/2 - e)] = 2 (79.68 \text{ k}) / [3 \text{ (8 ft)} (5.5 \text{ ft/2} - 1.20 \text{ ft})] \]

\[ = 4.28 \text{ ksf} < \text{SB}_{\text{allow}} = 5.9 \text{ ksf gross} \quad \text{OK} \quad \text{(this Practice, Equation 5)} \]

Stability ratio check using minimum axial load,

\[ \text{RM}_T = (\text{P}_{\text{min}})(b/2) = (71.71 \text{ kips})(5.5 \text{ ft} / 2) = 197.2 \text{ ft-k} \]

\[ \text{Stability ratio} = \text{RM}_T / \text{OTM}_T = 197.2 \text{ ft-k} / 95.47 \text{ ft-k} = 2.07 > 1.0 \quad \text{OK} \]

Check operating and transverse earthquake and eccentric (shell/fixed end):

Transverse operating earthquake load on exchangers,

0.7 \( V_{TEo} \) = 5.22 kips applied at the center of each exchanger

Axial loads at bottom of footing,

\[ \text{P}_{\text{max}} = P_s + P_o = (24.28 \text{ k}) + (37.0 \text{ k}) = 61.28 \text{ kips} \]

\[ (PIP STC01015, \text{ Table 5, Load Combination 3}) \]

\[ \text{P}_{\text{min}} = 0.9 \ (P_s + P_o) = (0.9)(24.28 \text{ k} + 37.0 \text{ k}) = 55.15 \text{ kips} \]

\[ (PIP STC01015, \text{ Table 5, Load Combination 5a}) \]

Transverse moment at bottom of footing,

\[ M_T = (5.22 \text{ kips}) \ [(2.75 \text{ ft} + 8.0 \text{ ft}) + (5.5 \text{ ft} + 2.75 \text{ ft} + 8.0 \text{ ft})] \ (0.4 \text{ shell end}) \]

\[ + (0.108)(4.54 \text{ k pier wt})(6.5 \text{ ft/2} + 1.5 \text{ ft}) + (5.72 \text{ ft-k pipe eccentricity}) \]

\[ = 64.43 \text{ ft-kips} \]
Soil-bearing check using maximum axial load,

\[
e = \frac{M_T}{P_{\text{max}}} = \frac{(64.43 \text{ ft-k})}{(61.28 \text{ k})} = 1.05 > \frac{b}{6} = \frac{5.5}{6} = 0.92
\]

\[
SB_{\text{max}} = \frac{2 P_{\text{max}}}{3 a (b/2 - e)} = \frac{2 (61.28 \text{ k})}{3 (8 \text{ ft}) (5.5 \text{ ft} / 2 - 1.05 \text{ ft})} = 3.00 \text{ ksf} < SB_{\text{allow}} = 5.9 \text{ ksf gross} \quad \text{OK (this Practice, Equation 5)}
\]

Stability ratio check using minimum axial load,

\[
RMT = \frac{(P_{\text{min}}) (b/2)}{(55.15 \text{ kips})(5.5 \text{ ft} / 2) = 151.7 \text{ ft-k}} \quad \text{OK}
\]

Check empty and transverse earthquake and eccentric (channel/sliding end):

Transverse empty earthquake load on exchangers,

\[
0.7 V_{\text{TTe}} = 3.79 \text{ kips} \quad \text{applied at the center of each exchanger}
\]

Minimum axial load at bottom of footing,

\[
P_{\text{min}} = 0.9 (P_s + P_e) = (0.9)(24.28 \text{ k} + 40.3 \text{ k}) = 58.12 \text{ kips} \quad \text{(PIP STC01015, Table 5, Load Combination 5b)}
\]

Transverse moment at bottom of footing,

\[
M_T = (3.79 \text{ kips}) [(2.75 ft + 8.0 ft) + (5.5 ft + 2.75 ft + 8.0 ft)] (0.6 \text{ channel end})
\]

\[
+ (0.108)(4.54 \text{ k pier wt})(6.5 ft/2 + 1.5 ft) + (6.24 \text{ ft-k pipe eccentricity})
\]

\[
= 69.97 \text{ ft-kips}
\]

Stability ratio check using minimum axial load,

\[
RMT = \frac{(P_{\text{min}}) (b/2)}{(58.12 \text{ kips})(5.5 \text{ ft} / 2) = 159.8 \text{ ft-k}} = 2.28 > 1.0 \quad \text{OK}
\]

Check empty and transverse earthquake and eccentric (shell/fixed end):

Transverse empty earthquake load on exchangers,

\[
0.7 V_{\text{TTe}} = 3.79 \text{ kips} \quad \text{applied at the center of each exchanger}
\]

Minimum axial load at bottom of footing,

\[
P_{\text{min}} = 0.9 (P_s + P_e) = (0.9)(24.28 \text{ k} + 26.9 \text{ k}) = 46.06 \text{ kips} \quad \text{(PIP STC01015, Table 5, Load Combination 5b)}
\]

Transverse moment at bottom of footing,

\[
M_T = (3.79 \text{ kips}) [(2.75 ft + 8.0 ft) + (5.5 ft + 2.75 ft + 8.0 ft)] (0.4 \text{ shell end})
\]

\[
+ (0.108)(4.54 \text{ k pier wt})(6.5 ft/2 + 1.5 ft) + (4.16 \text{ ft-k pipe eccentricity})
\]

\[
= 47.42 \text{ ft-kips}
\]

Stability ratio check using minimum axial load,

\[
RMT = \frac{(P_{\text{min}}) (b/2)}{(46.06 \text{ kips})(5.5 \text{ ft} / 2) = 126.7 \text{ ft-k}} \quad \text{OK}
\]
Check empty and bundle pull and eccentric (channel/sliding end; pulling top bundle out):

Vertical load from bundle pull on top exchanger,
\[ P_{bp} = (19.0 \text{ kips})(2.75 \text{ ft} + 5.5 \text{ ft}) / (11 \text{ ft}) = 14.25 \text{ kips} \]

Vertical load on sliding end at top of pier,
\[ P_{SL} = P_e + P_{bp} = (40.3 \text{ kips}) + (14.25 \text{ kips}) = 54.55 \text{ kips} \]

Horizontal load at sliding end,
\[ V_{SL} = \mu (P_{SL}) = (0.4)(54.55 \text{ kips}) = 21.82 \text{ kips} \]

Note that the horizontal load on the sliding end computed on the basis of friction is greater than half of the total bundle pull (19.0 kips). Therefore, because the two pedestals and footings are equal in size and thus even in stiffness, the actual horizontal load will be the same on both pedestals.

\[ V_{SL} = V_{TX} = 19.0 \text{ kips}/2 \text{ piers} = 9.5 \text{ kips} \]

Maximum axial load at bottom of footing,
\[ P_{max} = P_s + P_{SL} = (24.28 \text{ k}) + (54.55 \text{ k}) = 78.83 \text{ kips} \]

\[(PIP \ STC01015, \ Table \ 5, \ Load \ Combination \ 8)\]

Moments at bottom of footing,
\[ M_L = (V_{SL})(8.0 \text{ ft}) = (9.5 \text{ kips})(8.0 \text{ ft}) = 76.0 \text{ ft-kips} \]
\[ M_{Te} \text{ (from pipe eccentricity)} = 6.24 \text{ ft-kips} \]

Soil-bearing check using maximum axial load,
\[ e_1 = \frac{M_L}{P_{max}} = \frac{(76.0 \text{ ft-kips})}{(78.83 \text{ kips})} = 0.96 \text{ ft} \]
\[ e_2 = \frac{M_{Te}}{P_{max}} = \frac{(6.24 \text{ ft-kips})}{(78.83 \text{ kips})} = 0.08 \text{ ft} \]
\[ e_1 / a = (0.96 \text{ ft}) / (8 \text{ ft}) = 0.120 \]
\[ e_2 / b = (0.08 \text{ ft}) / (5.5 \text{ ft}) = 0.015 \]

Read Figure C, this Practice: \( K = 1.80 \)

\[ SB_{max} = K \left( \frac{P_{max}}{ab} \right) = (1.80) \left[ \frac{(78.83 \text{ k})}{(8 \text{ ft})(5.5 \text{ ft})} \right] = 3.23 \text{ ksf} < SB_{allow} = 5.9 \text{ ksf} \quad \text{OK} \]

Check empty and bundle pull and eccentric (shell/fixed end; pulling top bundle out):

Vertical load from bundle pull on top exchanger,
\[ P_{bp} = (19.0 \text{ kips})(2.75 \text{ ft} + 5.5 \text{ ft}) / (11 \text{ ft}) = 14.25 \text{ kips} \]

Vertical load on fixed end at top of pier,
\[ P_{FX} = P_e - P_{bp} = (26.9 \text{ kips}) - (14.25 \text{ kips}) = 12.65 \text{ kips} \]

Horizontal load at fixed end,
\[ V_{FX} = V_{SL} = 19.0 \text{ kips}/2 \text{ piers} = 9.5 \text{ kips} \]

Minimum axial load at bottom of footing,
\[ P_{min} = P_s + P_{FX} = (24.28 \text{ k}) + (12.65 \text{ k}) = 36.93 \text{ kips} \]

\[(PIP \ STC01015, \ Table \ 5, \ Load \ Combination \ 8)\]

Moments at bottom of footing,
\[
M_L = (V_{FX})(8.0 \text{ ft}) = (9.5 \text{ kips})(8.0 \text{ ft}) = 76.0 \text{ ft-kips}
\]
\[
M_{Te} \text{ (from pipe eccentricity)} = 4.16 \text{ ft-kips}
\]

Stability ratio check using minimum axial load,
\[
R_{ML} = P_{min} \left( a/2 \right) = (36.93 \text{ kips})(8.0 \text{ ft} / 2) = 148 \text{ ft-k}
\]
\[
\text{Stability ratio} = \frac{R_{ML}}{OT_{ML}} = \frac{148 \text{ ft-k}}{76.0 \text{ ft-k}} = 1.94 > 1.5 \text{ OK}
\]

- Use 8-ft by 5.5-ft by 1.50-ft footing.

**Footing Design**

Use load combinations and strength design load factors from *PIP STC01015*, Table 6.

Operating and thermal and eccentric (channel/sliding end):

- Load factors are from *PIP STC01015*, Table 6, Load Combination 1.

- Thermal force at top of pier,
  \[
  V_{u\text{Thermal}} = 1.4 \left( V_{\text{Thermal}} \right) = 1.4 (22.2 \text{ k}) = 31.08 \text{ kips}
  \]

- Axial load at bottom of footing,
  \[
  P_u = 1.4 \left( P_s + P_o \right) = 1.4 (24.28 \text{ k} + 55.4 \text{ k}) = 111.6 \text{ kips}
  \]

- Moments at bottom of footing,
  \[
  M_{ul} = (V_{u\text{Thermal}})(8 \text{ ft}) = (31.08 \text{ k})(8 \text{ ft}) = 248.6 \text{ ft-kips}
  \]
  \[
  M_{uTo} \text{ (from pipe eccentricity)} = 1.4 \left( M_{Te} \right) = 12.01 \text{ ft-kips}
  \]

- Maximum factored soil bearing,
  \[
  e_{u1} = \frac{M_{ul}}{P_u} = \frac{(248.6 \text{ ft-k})}{(111.6 \text{ k})} = 2.23 \text{ ft} > a/6 = (8 \text{ ft})/6 = 1.33 \text{ ft}
  \]
  \[
  e_{u2} = \frac{M_{uTo}}{P_u} = \frac{(12.01 \text{ ft-k})}{(111.6 \text{ k})} = 0.108 \text{ ft}
  \]

Because transverse eccentricity is very small, it can be ignored in calculations of factored soil bearing for design of footing reinforcing.

- Maximum factored soil bearing (longitudinal direction)
  \[
  S_{Bu\text{max}} = 2 \left( P_u \right) / (3b)(a/2 - e_{u1}) = 2(111.6 \text{ k}) / (3)(5.5 \text{ ft})(8 \text{ ft}/2 - (2.23 \text{ ft})] = 7.64 \text{ ksf}
  \]

- Calculate bearing length according to Equation 6, this Practice,
  \[
  \text{Bearing length (longitudinal direction)} = 3 \left( a/2 - e_{u1} \right) = 3 [(8 \text{ ft} / 2 - (2.23 \text{ ft})] = 5.31 \text{ ft}
  \]

- Factored soil bearing at face of pier (for checking moment),
  \[
  S_{Bu\text{face of pier}} = (7.64 \text{ ksf})(5.31 \text{ ft} - 3.33 \text{ ft}) / (5.31 \text{ ft}) = 2.85 \text{ ksf}
  \]

- Factored soil bearing at distance “d” from face of pier (for checking shear),
  \[
  d = (18\text{-inch footing}) - (3 \text{ inch clear}) - 1.5 \text{ (0.75-inch rebar)} = 13.87 \text{ inch} = 1.16 \text{ ft}
  \]
  \[
  S_{Bu\text{d from face of pier}} = (7.64 \text{ ksf})(5.31 \text{ ft} - 3.33 \text{ ft} + 1.16 \text{ ft}) / (5.31 \text{ ft}) = 4.52 \text{ ksf}
  \]
Operating and longitudinal earthquake and eccentric (shell/fixed end):

Load factors are from PIP STC01015, Table 6, Load Combination 3.

Longitudinal operating earthquake load on exchangers,

\[ V_{eo} = 12.10 \text{ kips} \]

Vertical load at top of piers from longitudinal operating earthquake load on exchangers (owing to overturning moment),

\[ P_{to} = (12.10 \text{ kips})(2.75 \text{ ft} + 8.25 \text{ ft}) / (11.0 \text{ ft}) = \pm 12.10 \text{ kips} \]

Maximum axial load at bottom of footing,

\[ P_u = 1.2 (P_s + P_o) + 1.0 (P_{to}) = 1.2 (24.28 \text{ k} + 37.0 \text{ k}) + 1.0 (12.10 \text{ k}) = 85.64 \text{ kips} \]

Moments at bottom of footing,

\[ M_{ul} = (0.7 \text{ at fixed end})(12.10 \text{ k})(2 \text{ exchangers})(8.0 \text{ ft}) \]
\[ + (0.250)(4.54 \text{ k pier wt})(6.5 \text{ ft/2} + 1.5 \text{ ft}) \]
\[ = 140.91 \text{ ft-kips} \]

\[ M_{uTo} \text{ (from pipe eccentricity)} = 1.2 (M_{To}) = 1.2 (5.72 \text{ ft-kips}) = 6.86 \text{ ft-kips} \]

Maximum factored soil bearing,

\[ c_{u1} = M_{ul} / P_u = (140.91 \text{ ft-k}) / (85.64 \text{ k}) = 1.65 \text{ ft} > a/6 = (8 \text{ ft})/6 = 1.33 \text{ ft} \]

\[ c_{u2} = M_{uTo} / P_u = (6.86 \text{ ft-k}) / (85.64 \text{ k}) = 0.080 \text{ ft} \]

Because transverse eccentricity is very small, it can be ignored in calculations of factored soil bearing for design of footing reinforcing.
Calculate bearing length according to Equation 6, this Practice,

$S_{Bu_{\text{face of pier}}} = (4.42 \text{ ksf})(7.05 \text{ ft} - 3.33 \text{ ft}) / (7.05 \text{ ft}) = 2.33 \text{ ksf}$

Factored soil bearing at distance “d” from face of pier (for checking shear),

$d = (18\text{-inch footing}) - (3 \text{ inch clear}) - 1.5 (0.75\text{-inch rebar}) = 13.87 \text{ inch} = 1.16 \text{ ft}$

$S_{Bu_{d \text{ from face of pier}}} = (4.42 \text{ ksf})(7.05 \text{ ft} - 3.33 \text{ ft} + 1.16 \text{ ft}) / (7.05 \text{ ft}) = 3.06 \text{ ksf}$

Empty and bundle pull and eccentric (channel/sliding end; pulling top bundle out):

Load factors are from $PIP \ STC01015$, Table 6, Load Combination 9

Vertical load from bundle pull on top exchanger,

$P_{b_p} = (19.0 \text{ kips})(2.75 \text{ ft} + 5.5 \text{ ft}) / (11 \text{ ft}) = 14.25 \text{ kips}$

Horizontal load at sliding end,

$V_{uSL} = V_{uFX} = 1.6 (19.0 \text{ kips/2 piers}) = 15.2 \text{ kips}$

Maximum axial load at bottom of footing,

$P_u = 1.2 (P_s + P_e) + 1.6 (P_{b_p}) = 1.2 (24.28 \text{ k} + 40.3 \text{ k}) + 1.6 (14.25 \text{ k}) = 100.3 \text{ kips}$

Moments at bottom of footing,

$M_{ul} = (V_{uSL})(8.0 \text{ ft}) = (15.2 \text{ kips})(8.0 \text{ ft}) = 121.6 \text{ ft-kips}$

$M_{Te} \text{ (from pipe eccentricity)} = 1.2 (M_{Te}) = 1.2 (6.24 \text{ ft-kips}) = 7.49 \text{ ft-kips}$

Maximum and minimum factored soil bearing,

$e_{u1} = M_{ul} / P_u = (121.6 \text{ ft-kips}) / (100.3 \text{ kips}) = 1.21 \text{ ft} \leq a/6 = (8 \text{ ft})/6 = 1.33 \text{ ft}$
Because transverse eccentricity is very small, it can be ignored in calculations of factored soil bearing for design of footing reinforcing.

\[
S_{Bu} = \left( \frac{P_u}{A} \right) [1 \pm (6)(e_u/a)]
\]

\[
S_{Bu_{\text{max}}} = \frac{[(100.3 \text{ k}) / (8 \text{ ft})(5.5 \text{ ft})]}{[1 + (6)(1.21 \text{ ft})/(8 \text{ ft})]} = 4.35 \text{ ksf}
\]

\[
S_{Bu_{\text{min}}} = \frac{[(100.3 \text{ k}) / (8 \text{ ft})(5.5 \text{ ft})]}{[1 - (6)(1.21 \text{ ft})/(8 \text{ ft})]} = 0.21 \text{ ksf}
\]

Factored soil bearing at face of pier (for checking moment),

\[
S_{Bu_{\text{face of pier}}} = (0.21 \text{ ksf}) + (4.35 \text{ ksf} - 0.21 \text{ ksf})(8 \text{ ft} - 3.33 \text{ ft})/(8 \text{ ft}) = 2.63 \text{ ksf}
\]

Factored soil bearing at distance “d” from face of pier (for checking shear),

\[
d = (18\text{-inch footing}) - (3\text{-inch clear}) - 1.5(0.75\text{-inch rebar}) = 13.87 \text{ inch} = 1.16 \text{ ft}
\]

\[
S_{Bu_{d \text{ from face of pier}}} = (2.63 \text{ ksf}) + (4.35 \text{ ksf} - 0.21 \text{ ksf})(1.16 \text{ ft})/(8 \text{ ft}) = 3.23 \text{ ksf}
\]

By comparison, operating and thermal and eccentric (channel/sliding end) will govern the design.

Factored soil and concrete weight,

\[
w_u = (1.4) [(0.15 \text{ kcf})(1.5\text{-ft footing}) + (0.10 \text{ kcf})(2.5\text{-ft soil})] = 0.67 \text{ ksf}
\]

Factored shear at a distance “d” from face of pier,

\[
V_u = (4.52 \text{ ksf} - 0.67 \text{ ksf})(3.33 \text{ ft} - 1.16 \text{ ft}) + (7.64 \text{ ksf} - 4.52 \text{ ksf})(3.33 \text{ ft} - 1.16 \text{ ft})/2
\]

\[
= 8.35 \text{ kips} + 3.39 \text{ kips}
\]

\[
= 11.74 \text{ kips (per ft width)}
\]

Factored moment at face of pier,

\[
M_u = (2.85 \text{ ksf} - 0.67 \text{ ksf})(3.33 \text{ ft})^2(1/2) + (7.64 \text{ ksf} - 2.85 \text{ ksf})(3.33 \text{ ft})^2(1/3)
\]
= 12.09 ft-k + 17.70 ft-k
= 29.79 ft-k (per ft width)

Check diagonal tension shear (at a distance “d” from face of pier):

\[ \phi V_c = \phi \cdot 2 \sqrt{f'c} \cdot b_w \cdot d \]  
(ACI 318-05, Equation 11-3)

\[ = (0.75)(2) \sqrt{4,000 \text{ psi}} (12 \text{ inches})(13.87 \text{ inches}) / 1,000 \]

\[ = 15.79 \text{ kips (per ft width)} > V_u \quad \text{OK} \]

Check for two-way action (punching shear) according to ACI 318-05, Section 11.12 (at distance “d/2” from face of pier):

By inspection, operating weight at channel end will govern.

\[ P_u = 1.4 \left( P_s + P_o \right) = 1.4 \left[ (0.15 \text{ kcf})(3.50 \text{ ft})(1.33 \text{ ft})(6.5 \text{ ft}) + (55.4 \text{ kips}) \right] = 83.9 \text{ kips} \]

\[ S_{Bu} = P_u / (ab) = (83.9 \text{ kips}) / (8 \text{ ft})(5.5 \text{ ft}) = 1.91 \text{ ksf} \]

\[ V_u = P_u - \left( S_{Bu} \right)(3.5 \text{ ft} + d)(1.33 \text{ ft} + d) \]

\[ = 83.9 \text{ kips} - (1.91 \text{ ksf})(3.5 \text{ ft} + 1.16 \text{ ft})(1.33 \text{ ft} + 1.16 \text{ ft}) = 61.7 \text{ kips} \]

\[ \beta = (3.5 \text{ ft})/(1.33 \text{ ft}) = 2.63 \quad b_o = 2(3.5 \text{ ft} + 1.16 \text{ ft}) + 2(1.33 \text{ ft} + 1.16 \text{ ft}) = 14.3 \text{ ft} \]

\[ \alpha_s = 40 \]

\[ V_c = (2 + 4/\beta) \sqrt{f'c} \ b_o \ d \]

\[ = (2 + 4/2.63) \sqrt{4,000 \text{ psi}} (14.3 \text{ ft})(12 \text{ inches/ft})(13.87 \text{ inches})/1,000 \]

\[ = 530 \text{ kips} \]  
(ACI 318-05, Equation 11-33)

\[ V_c = (\alpha_s d/b_o + 2) \sqrt{f'c} \ b_o \ d \]

\[ = [(40)(1.16 \text{ ft})/(14.3 \text{ ft}) + 2] \sqrt{4,000 \text{ psi}} (14.3 \text{ ft})(12 \text{ inches/ft})(13.87 \text{ inches})/1,000 \]

\[ = 790 \text{ kips} \]  
(ACI 318-05, Equation 11-34)

\[ V_c = 4 \sqrt{f'c} \ b_o \ d = 4 \sqrt{4,000 \text{ psi}} (14.3 \text{ ft})(12 \text{ inches/ft})(13.87 \text{ inches})/1,000 = 602 \text{ kips} \]

(ACI 318-05, Equation 11-35)

ACI 318-05, Equation 11-33, governs, \( V_c = 530 \text{ kips} \).

\[ V_n = \phi V_c = 0.75(530 \text{ kips}) = 397.5 \text{ kips} > V_u = 61.7 \text{ kips} \quad \text{OK} \]

Design for moment:

\[ F = b \ d^2 / 12,000 = (12 \text{ inches})(13.87 \text{ inches})^2 / 12,000 = 0.192 \]

\[ K_u = M_u / F = (29.79 \text{ ft-k}) / (0.192) = 155.2 \rightarrow \rho = 0.0030 \]

\[ A_r = \rho \ b \ d = (0.0030)(12 \text{ inches})(13.87 \text{ inches}) = 0.50 \text{ inches}^2/\text{ft} \quad \text{controls} \]

\[ A_{r \text{min}} (\text{for temperature reinforcing}) = (0.0018)(\text{gross concrete area}) \]

\[ = (0.0018)(12 \text{ inches})(18 \text{ inches}) \]

\[ = 0.39 \text{ inches}^2/\text{ft} \]  
(ACI 318-05, Section 7.12.2)
Note that *ACI 318-05*, Section 10.5.1, does not apply because Section 10.5.4 excludes foundations of uniform thickness from the minimum reinforcing requirements of Section 10.5.1.

- Use #6 at 10 inch each way ($A_s = 0.53$ inch$^2$/ft).

**Top steel:**

Because the bottom of the foundation is not in full bearing for some loading combinations and because the footing is designed for earthquake loads, ductility is required; therefore, the top of the foundation mat needs to be reinforced.

Conservatively calculate moment for top steel considering the weight of soil and concrete with a 1.4 load factor and assuming no soil bearing under the portion of the footing extending from the edge of the pier.

Factored soil and concrete weight,

$$w_u = (1.4) [(0.15 \text{ kcf})(1.5\text{-ft footing}) + (0.10 \text{ kcf})(2.5\text{-ft soil})] = 0.67 \text{ ksf}$$

Factored shear at a distance “$d_{top}$” from face of pier,

$$d_{top} = (18\text{-inch footing}) - (2\text{-inch clear}) - 1.5(0.50\text{-inch rebar}) = 15.25 \text{ inch} = 1.27 \text{ ft}$$

$$V_u = (0.67 \text{ ksf})(3.33 \text{ ft} - 1.27 \text{ ft}) = 1.38 \text{ kips (per ft width)}$$

Factored moment at face of pier,

$$M_u = (0.67 \text{ ksf})(3.33 \text{ ft})^2(1/2) = 3.72 \text{ ft-k (per ft width)}$$

Design for moment:

$$F = b (d_{top})^2 / 12,000 = (12 \text{ inches})(15.25 \text{ inches})^2 / 12,000 = 0.233$$

$$K_u = M_u / F = (3.72 \text{ ft-k}) / (0.233) = 16.0 \rightarrow \rho = 0.0013$$

$$A_s = \rho b d_{top} = (0.0013)(12 \text{ inches})(15.25 \text{ inches}) = 0.24 \text{ inches}^2$/ft $\leftarrow$ controls

- Use #4 at 10 inches each way ($A_s = 0.24$ inch$^2$/ft).
Example 2 - Horizontal Vessel Foundation

**Plan**

Dimensions typical both piers

- Bearing plate
  - 11 ft -9 inches by 9 inches by 3/8 inch

- Low-friction manufactured slide assembly:
  - (7 components with 1 upper element and 1 lower element per component)
  - Upper elements = 11 inches by 3 1/2 inches
  - Lower elements = 8 inches by 2 1/2 inches

**Section "A - A"**

- 2 - 1 1/4 inch diameter
  - ASTM F1554, Grade 36
  - anchor bolts per pier
  - P = 4 inches (fixed end w/1 nut)
  - P = 5 1/4 inches (sliding end w/2 nuts)

- #4 ties @ 12 inches
- 13 #5 bars each face with matching dowels
- #6 @ 12 inches each way

- Top of grout elevation (fixed end)
- Top of low-friction manufactured slide assembly (sliding end)
(Example 2, continued)

**DESIGN DATA**

**Vessel Data:**
- Empty weight = 98 kips
- Operating weight = 335 kips
- Test weight = 394 kips
- Basic diameter = 12 ft
- Maximum design temperature = 500°F
- Vessel material: carbon steel
- Bolts: two 1-1/4-inch diameter, *ASTM F1554*, Grade 36 (galvanized) per pier
- Bolt spacing: 11 ft-0 inch
- Saddle: 11 ft-8 inches by 10 inches

**Design Criteria:**
- Concrete: \( f'c = 4,000 \text{ psi} \)
- Reinforcing: \( f_y = 60,000 \text{ psi} \)
- Soil unit weight: \( \gamma = 100 \text{ pcf} \)
- Allowable net soil-bearing: \( SB_{net} = 3.8 \text{ ksf} \) (at 4-ft depth)
- Wind loads: *ASCE/SEI 7-05*
- Earthquake loads: *ASCE/SEI 7-05*
- Use a 20% increase in soil allowable pressure for test load combinations.

**DETERMINE LOADS**

**Empty, Operating, and Test Loads**
Include an additional 10% of the applicable weight (empty, operating, or test) to account for piping supported on the horizontal vessel (refer to this Practice, Section 4.2, “Vertical Loads”):
- Total empty load, \( D_e = (1.10)(98 \text{ kips}) = 107.8 \text{ kips} \)
- Total operating load, \( D_o = (1.10)(335 \text{ kips}) = 368.5 \text{ kips} \)
- Total test load, \( D_t = (1.10)(394 \text{ kips}) = 433.4 \text{ kips} \)

**Transverse Moment from Pipe Eccentricity**
- Eccentricity = (basic diameter)/2 + (1.5 ft) = (12 ft)/2 + (1.5 ft) = 7.5 ft
- Empty transverse moment per pier, \( M_{Te} = (98 \text{ kips})(0.1)(0.5 \text{ per pier})(7.5 \text{ ft}) = 36.8 \text{ ft-k} \)
- Operating transverse moment per pier, \( M_{To} = (335 \text{ kips})(0.1)(0.5 \text{ per pier})(7.5 \text{ ft}) = 125.6 \text{ ft-k} \)
- Test transverse moment per pier, \( M_{Tt} = (394 \text{ kips})(0.1)(0.5 \text{ per pier})(7.5 \text{ ft}) = 147.8 \text{ ft-k} \)

**Wind Loads:**
- Wind load calculations are beyond the scope of this Practice.
- Vessel wind is applied at the center of the vessel.
Transverse wind: $H_w = 13.44$ kips
Longitudinal wind: $H_w = 2.85$ kips
Transverse or longitudinal wind on each pier: $H_w = 0.076$ ksf

**Thermal Force:**

1. Compute sliding force (assume that a low-friction manufactured slide plate assembly is used):
   \[ \mu = 0.10 \text{ (maximum based on manufacturer’s literature)} \]
   Operating load, $P_o = (368.5 \text{ kips}) / (2 \text{ piers}) = 184.3 \text{ kips}$
   \[ F_f = \mu (P_o) = (0.10)(184.3 \text{ kips}) = 18.43 \text{ kips} \quad \text{(Equation 1, this Practice)} \]

2. Compute force required to deflect pier:
   Assume pier is 12 ft -2 inches long by 14 inches wide by 81 inches high:
   Moment of inertia, $I = b (h)^3 / 12 = (146 \text{ inches})(14 \text{ inches})^3 / 12 = 33,385 \text{ inches}^4$
   Modulus of elasticity, $E_c = 57,000 \sqrt{f’c} = 57,000 \sqrt{4,000 \text{ psi}} = 3,605 \text{ ksi}$
   (ACI 318-05, Section 8.5.1)
   Thermal expansion coefficient for carbon steel at 500°F: $\varepsilon = 0.0362 \text{ inches/ft}$
   (this Practice, Table 1)
   Thermal growth between saddles, $\Delta = (\varepsilon)(L) = (0.0362 \text{ inches/ft})(22 \text{ ft}) = 0.796 \text{ inches}$
   \[ T = 3 \Delta E I / 2 H^3 = 3 (0.796 \text{ inches})(3,605 \text{ ksi})(33,385 \text{ inches}^4) / 2 (81 \text{ inches})^3 = 270.4 \text{ kips} \quad \text{(this Practice, Equation 2)} \]

   - Because $F_f < T$, and to reduce high-friction forces, use a low-friction manufactured slide plate assembly.

**DESIGN ELEMENTS**

**Size Low-Friction Manufactured Slide Plate Elements**
Upper element width = (saddle width) + 1 inch = (10 inches) + 1 inch = 11 inches
Lower element width = upper element width - 2 (Δ) - 1 inch
= (11 inches) - (2)(0.796 inches) - 1 inch ≈ 8 inches

Maximum load on sliding end (from test weight): $P_t = (433.4 \text{ kips}) / (2 \text{ piers}) = 216.7 \text{ kips}$
Operating load on sliding end: $P_o = (368.5 \text{ kips}) / (2 \text{ piers}) = 184.3 \text{ kips}$

According to manufacturer’s recommendations, seven slide plate components are required for each assembly, with the lower element = 8 inches by 2.5 inches and the upper element = 11 inches by 3.5 inches.

Total length of lower elements provided = 7(2.5 inches) = 17.5 inches
Maximum bearing pressure on elements = $(216.7 \text{ k}) / [(7)(8 \text{ inches})(2.5 \text{ inches})] = 1,548 \text{ psi}$
Operating bearing pressure on elements = $(184.3 \text{ k}) / [(7)(8 \text{ inches})(2.5 \text{ inches})] = 1,316 \text{ psi}$

From manufacturer’s literature for 1,316 psi bearing: $\mu = 0.055$
Revised operating frictional force = $F_f = (0.055)(184.3k) = 10.14 \text{ kips}$
Size Steel Bearing Plate

Steel bearing plate dimensions:

Width = (lower slide plate element width) + (1 inch) = (8 inches) + (1 inch) = 9 inches

Length = (saddle length) + (1 inch) = (11 ft - 8 inches) + (1 inch) = 11 ft - 9 inches

Check bearing stress (test load case):

\[ P_u = 1.4 \times P_t = 1.4 \times \left( \frac{433.4 \text{ kips}}{2 \text{ piers}} \right) = 303.4 \text{ kips} \]

\[ P_n = \phi 0.85 f'c A_1 = (0.65)(0.85)(4 \text{ ksi})(9 \text{ inches})(141 \text{ inches}) = 2,804 \text{ kips}, \quad \text{OK} \]

(ACI 318-05, Section 10.17)

- Use a bearing plate 11 ft - 9 inches by 9 inches by 3/8 inches.

Pier Size

Pier length:

\[ (\text{c/c bolts}) + (2)(5\text{-inch minimum anchor bolt edge distance}) = (11 \text{ ft - 0 inches}) + 2 (5 \text{ inches}) \]

= 11 ft - 10 inches

(bearing plate length) + (4 inches) = (11 ft - 9 inches) + (4 inches)

= 12 ft - 1 inch \quad \leftarrow \text{controls,}

but use 12 ft - 2 inches for forming in 2-inch increments

Pier width:

10 inches

10\% of pier height = (0.10)(81 inches) = 8.1 inches (based on assumed pier height)

(2)(5\text{-inch minimum anchor bolt edge distance}) = 2 (5 \text{ inches}) = 10 \text{ inches}

(bearing plate width) + (4 inches) = (10 \text{ inches}) + (4 inches)

= 14 \text{ inches} \quad \leftarrow \text{controls}

- Use a pier size of 1 ft - 2 inches by 12 ft - 2 inches.

Anchor Bolt Design, Pier Design, and Footing Design

Anchor bolt design, pier design, and footing design are very similar to Example 1. Example 1 should be followed for these portions of this example.