

Process Industry Practices  
Structural

**PIP STE03350**  
**Vertical Vessel Foundation**  
**Design Guide**

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## 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.

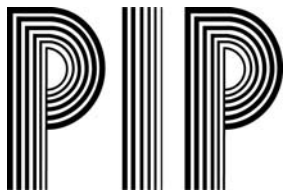
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## Process Industry Practices Structural

# PIP STE03350 Vertical Vessel Foundation Design Guide

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## 1. Introduction

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### 1.1 Purpose

This Practice provides guidelines and recommended procedures for engineers analyzing and designing vertical vessel foundations.

### 1.2 Scope

This Practice addresses isolated foundations supported directly on soil. Pile-supported footings are not included in this Practice.

## 2. References

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Applicable requirements of the latest edition of the following guides, standards, and regulations in effect on the date of contract award should be considered an integral part of this Practice, except as otherwise noted. Short titles are used herein where appropriate.

### 2.1 Process Industry Practices (PIP)

- PIP STC01015 - *Structural Design Criteria*
- PIP STE05121 - *Anchor Bolt Design Guide*
- PIP STE03360 - *Heat Exchanger and Horizontal Vessel Foundation Design Guide*
- PIP STF05121 - *Fabrication and Installation of Anchor Bolts*

### 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)
  - ASCE/SEI 7-05 - *Minimum Design Loads for Buildings and Other Structures*
- ASTM International (ASTM)
  - ASTM F1554 - *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength*

## 3. Definitions

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For the purposes of this Practice, the following definitions apply:

*engineer*: The engineer who performs the structural design

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

*owner:* The party who has authority through ownership, lease, or other legal agreement over the site, facility, structure or project wherein the foundation will be constructed

## 4. Design Procedure

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### 4.1 Design Considerations

- 4.1.1 The engineer should review and verify project design as based on applicable codes, corrosion allowances for anchor bolts, anchor bolt types, and any special requirements dictated by the owner.
- 4.1.2 For very tall or heavy vessels, sufficient capacity cranes may not be available for erection. The engineer should determine whether additional loading may be imposed on the foundation during erection.

### 4.2 Vertical Loads

#### 4.2.1 Dead Loads

- 4.2.1.1 The following nominal loads should be considered as dead loads when applying load factors used in strength design.
  - a. Structure dead load ( $D_s$ ) - Weight of the foundation and weight of the soil above the foundation that are resisting uplift. Pedestal dead load ( $D_p$ ) is a part of  $D_s$  representing the weight of the pedestal used in the calculation of tension in pedestal dowels.
  - b. Erection dead load ( $D_f$ ) - Fabricated weight of the vessel generally taken from the certified vessel drawing
  - c. Empty dead load ( $D_e$ ) - Empty weight of the vessel, including all attachments, trays, internals, insulation, fireproofing, agitators, piping, ladders, platforms, etc.
  - d. Operating dead load ( $D_o$ ) - Empty dead load of the vessel plus the maximum weight of contents (including packing/catalyst) during normal operation
  - e. Test dead load ( $D_t$ ) - 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 the test medium. Cleaning load should be used for test dead load if cleaning fluid is heavier than test medium. Whether to test or clean in the field should be determined. Designing for test dead load is generally desirable because unforeseen circumstances may occur.
- 4.2.1.2 Eccentric vessel loads caused by large pipes or reboilers should be considered for the applicable load cases.

#### **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 as listed in Tables 3 and 4 of *PIP STC01015* will typically not control any part 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.

*Comment:* Loads from vendor or other engineering disciplines without verification should not be accepted.

4.3.1.3 When calculating or checking wind loads, due consideration should be given to factors that may significantly affect total wind loads, such as the application of dynamic gust factors or the presence of spoilers, platforms, ladders, piping, etc., on the vessel.

#### **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.

*Comment:* Loads from vendor or other engineering disciplines should not be accepted without verification.

4.3.2.3 For skirt-supported vertical vessels classified as Occupancy Category IV in accordance with *ASCE/SEI 7-05*, Section 1.5.1 and Table 1-1, the critical earthquake provisions and implied load combination of *ASCE/SEI 7-05*, Section 15.7.10.5, should be followed.

#### **4.3.3 Other Loading**

4.3.3.1 Thrust forces caused by thermal expansion of piping should be included in the calculations for operating load combinations, if deemed advisable. The pipe stress engineer should be consulted for any thermal loads that are to be considered.

4.3.3.2 Consideration should be given to process upset conditions that could occur and could increase loading on the foundation.

### **4.4 Load Combinations**

The vertical vessel foundation should be designed using load combinations in accordance with *PIP STC01015*.

## 4.5 Pedestal

4.5.1 Concrete pedestal dimensions should be sized on the basis of standard available forms for the project. When form information is not available, octagon pedestal dimensions should be sized with pedestal faces in 2-inch increments to allow use of standard manufactured forms. The following criteria should be used to determine the size and shape for the pedestal.

4.5.1.1 Face-to-face pedestal size should be no less than the largest of the following:

$$BC + 9 \text{ inches} \quad (\text{Equation 1a})$$

$$BC + 8 \text{ (BD)} \text{ (for Grade 36 anchor bolts)} \quad (\text{Equation 1b})$$

$$BC + 12 \text{ (BD)} \text{ (for high-strength anchor bolts)} \quad (\text{Equation 1c})$$

$$BC + SD + 9 \text{ inches} - \text{(BD)} \quad (\text{Equation 1d})$$

$$BC + SD + 7 \text{ (BD)} \text{ (for Grade 36 anchor bolts)} \quad (\text{Equation 1e})$$

$$BC + SD + 11 \text{ (BD)} \text{ (for high-strength anchor bolts)} \quad (\text{Equation 1f})$$

where:

BC = bolt circle, inches

BD = bolt diameter, inches

SD = sleeve diameter, inches

4.5.1.2 Pedestals 6 ft and larger should be octagonal. Dimensions for octagon pedestals are provided in Table 1. Octagons highlighted in gray in Table 1 have faces in 2-inch increments.

4.5.1.3 Pedestals smaller than 6 ft should be square, or round if forms are available.

4.5.2 Anchorage - It is normally desirable to make the pedestal deep enough to contain the anchor bolts and to keep them out of the footing. Consideration should be given to anchor bolt development and foundation depth requirements. Pedestal size may need to be increased to provide adequate  $A_N$  for anchor bolts when additional reinforcement for anchor bolts is not used.

4.5.3 Pedestal reinforcement - The pedestal should be tied to the footing with sufficient dowels around the pedestal perimeter to prevent separation of the pedestal and footing. Development of reinforcing steel should be checked.

4.5.4 Dowels - Dowels should be sized by computing the maximum tension existing at the pedestal perimeter attributable to overturning moments. Conservatively, the following formulas may be used. More exact tension loads may be obtained by using *ACI 318* strength design methodology.

$$\text{Tension, } F_u = 4(M_{u_{\text{ped}}}) / [(N_d)(DC)] - 0.9[(D_e \text{ or } D_o) + D_p] / N_d$$

(Equation 2)

$(D_e \text{ or } D_o) =$  nominal empty or operating vessel weight. Use empty weight for wind loads. Use empty or operating for earthquake loads depending on which condition is used to calculate  $M_{u_{ped}}$ .

$$A_s \text{ required} = \text{tension/design stress} = F_u / (\phi f_y) \quad (\text{Equation 3})$$

where:

$F_u$  = maximum ultimate tension in reinforcing bar

$M_{u_{ped}}$  = maximum factored overturning moment at base of pedestal, calculated by using load factors in load combinations for uplift cases in Table 4 ("Loading Combinations and Load Factors - Strength Design") of *PIP STC01015*

$N_d$  = number of dowels (assumed); should be a multiple of 8

DC = dowel circle diameter (assume pedestal size minus 6 inches)

$D_e + D_p$  = nominal empty weight of vessel and pedestal weight

$D_o + D_p$  = nominal operating weight of vessel and pedestal weight

$\phi$  = strength reduction factor = 0.90

$f_y$  = yield strength of reinforcing steel

4.5.5 Minimum pedestal reinforcement should be as follows:

Octagons 6 ft - 0 inch to 8 ft - 6 inches:

16, #4 verticals with #3 ties at 15-inch maximum

Octagons larger than 8 ft - 6 inches to 12 ft - 0 inch:

24, #5 verticals with #4 ties at 15-inch maximum

Octagons larger than 12 ft - 0 inch:

#5 verticals at 18-inch maximum spacing with #4 ties at 15-inch maximum

4.5.6 Top reinforcement - A mat of reinforcing steel at the top of the pedestal should be provided. Minimum steel should be #4 bars at 12-inch maximum spacing across the flats in two directions only.

4.5.7 Ties - See minimum pedestal reinforcement, Section 4.5.5, this Practice.

## 4.6 Anchor Bolts

See *PIP STE05121* for anchor bolt design procedures. The nomenclature in the following sections is from *PIP STE05121*.

4.6.1 Conservatively, the maximum tension on an anchor bolt may be determined using the following formula. More exact tension loads may be obtained by using *ACI 318* strength design methodology.



$$N_u = 4M_u / [(N_b)(BC)] - 0.9(D_e \text{ or } D_o) / N_b \quad (\text{Equation 4})$$

where:

$N_u$  = factored maximum tensile load on an anchor bolt

$M_u$  = factored moment at the base of the vessel, calculated using load factors in load combinations for uplift cases in Table 4 ("Loading Combinations and Load Factors - Strength Design") of *PIP STC01015*

$N_b$  = number of anchor bolts

BC = bolt circle diameter

$(D_e \text{ or } D_o)$  = nominal empty or operating vessel weight. Use empty weight for wind loads. Use empty or operating for earthquake loads, depending on which condition is used to calculate  $M_u$ .

- 4.6.2 For most cases, there is no shear on the anchor bolts because the load is resisted by friction caused primarily by the overturning moment. If friction cannot resist the load, the bolts should be designed to resist the entire shear load, or other methods may be used to resist the shear load. The friction resistance can be calculated using the following formulas:

$$P_u = M_u / LA + 0.9(D_e \text{ or } D_o) / 2 \quad (\text{Equation 5})$$

$$V_f = \mu P_u \quad (\text{Equation 6})$$

where:

$P_u$  = factored compression force at top of pedestal

LA = lever arm between centroid of tension loads on bolts and the centroid of the compression load on the pedestal. This is a complicated distance to determine exactly. A conservative approximation is to use 2/3 of the bolt circle diameter as the lever arm.

$\mu$  = coefficient of friction. For the normal case of grout at the surface of the pedestal,  $\mu = 0.55$ .

$V_f$  = frictional resisting force (factored)

$$\text{To have no shear load on the bolts: } V_u \leq \phi V_f \quad (\text{Equation 7})$$

where:

$V_u$  = factored shear load at base of vessel, calculated using load factors in load combinations for uplift cases in Table 4 ("Loading Combinations and Load Factors - Strength Design") of *PIP STC01015*

$\phi$  = strength reduction factor = 0.75

- 4.6.3 Exact formulas for  $A_N$  (the projected concrete failure area for an anchor or group of anchors for calculation of concrete strength in tension) in an octagon foundation are fairly complex. To determine this area, it may be easier to perform graphical calculations or to use CAD programs. Use of an equivalent circle, which is a circle with the same area as the octagon, may

help to approximate  $A_N$ . The diameter of an equivalent circle is  $1.027 D$ , where  $D$  is the distance across the flats of the octagon. Figure A shows potential  $A_N$ s for various configurations. Note that alternate configurations exist for the same general pattern. The engineer may also take advantage of group action and use the average load on a group of bolts over a larger  $A_N$ . The engineer may use the configuration that provides the largest  $A_N$ .

- 4.6.4 The  $N_u$  and the  $A_N$  determined according to this section should be used along with *PIP STE05121* to design the anchor bolts. For design work, PIP Member Companies may use the *PIP Anchor Bolt Design Spreadsheet*, available in the Implementation Resource Center on the Member Area of the PIP web site, under the tab "TOOLS."

## 4.7 Footing Design

### 4.7.1 Sizing

Footings for vertical vessels may be octagonal or square and sized based on standard available form sizes. When form information is not available, footing dimensions should be sized with footing faces in 2-inch increments to allow use of standard manufactured forms. (Octagons highlighted in gray in Table 1 are those having faces in 2-inch increments.) If extended to the recommended depth specified in the geotechnical report, the pedestal may be adequate without a footing. Footings smaller than 7 ft - 0 inch in diameter should be square.

Where a footing is required, the footing thickness should be a minimum of 12 inches. The footing thickness should be adequate to develop pedestal reinforcement and to satisfy the shear requirements of *ACI 318*.

The footing thickness should also be checked for top tension without top reinforcement in accordance with *ACI 318*. If the thickness is not adequate, either a thicker footing or top reinforcing steel is required (see Section 4.7.5). Note that increasing the footing thickness is typically more cost effective for construction than adding a top mat of reinforcing steel except where seismic effects create tensile stresses requiring top reinforcement.

For the first trial, the diameter of an octagonal footing may be approximated by the following formula:

$$\text{Diameter} = (2.6)(M_{\text{ftg}}/\text{SB})^{1/3} \quad (\text{Equation 8})$$

where:

$M_{\text{ftg}}$  = nominal overturning moment at base of footing, kip ft

SB = allowable gross soil bearing, ksf

#### 4.7.2 Soil Bearing - Octagon

- 4.7.2.1. Soil-bearing pressure should be checked for maximum allowable pressure on the diagonal.
- 4.7.2.2 Soil-bearing pressure used for footing design should be computed on the flat.
- 4.7.2.3 Where the total octagonal footing area is in compression ( $e/D \leq 0.122$  on the diagonal and  $e/D \leq 0.132$  on the flat), the soil-bearing pressure should be computed using the following formulas:

$$f = (P/A) \pm (M_{ftg}/S) \quad (\text{Equation 9})$$

$$f (\text{diagonal}) = P/A [1 \pm (8.19e/D)] \quad (\text{Equation 10a})$$

$$f (\text{flat}) = P/A [1 \pm (7.57e/D)] \quad (\text{Equation 10b})$$

where:

D = distance between parallel sides, ft

f = toe pressure, ksf

P = nominal total vertical load including soil and foundation, kip

A = bearing area of octagonal footing ( $0.828D^2$ ),  $\text{ft}^2$

$M_{ftg}$  = nominal overturning moment at base of footing, -kip ft

S = section modulus,  $\text{ft}^3$

e = eccentricity ( $M_{ftg}/P$ ), ft

- 4.7.2.4 Where the total octagonal footing area is not in compression ( $e/D > 0.122$  on the diagonal and  $e/D > 0.132$  on the flat), the soil-bearing pressure should be computed using Figure B and the following formula:

$$f = LP/A \quad (\text{Equation 11})$$

where the value of L is obtained from Figure B.

- 4.7.2.5 The  $e/D$  ratios for octagon footings may go above the limits of the chart in Figure B because of the load factors in strength design. L and K values for these conditions are tabulated in Table 2 for lateral loads perpendicular to a face. These values shall be used for calculating moments and shears in the footing. They should not be used to check soil-bearing pressures.

#### 4.7.3 Soil Bearing - Square

- 4.7.3.1 Where the total footing is in compression ( $e/b \leq 0.167$ ), the soil-bearing pressure should be computed using the following formula:

$$f = P/A (1 \pm 6e/b) \quad (\text{Equation 12})$$

where:

b = dimension of footing in the direction of the overturning moment, ft

A = bearing area of square footing,  $\text{ft}^2$

- 4.7.3.2 Where the total footing is not in compression ( $e/b > 0.167$ ), the soil-bearing pressure should be computed using the following formula:

$$f = P/A [4b/(3b-6e)] \quad (\text{Equation 13})$$

$$f = 0 \text{ at } 3(b/2-e) \text{ from edge of footing} \quad (\text{Equation 14})$$

- 4.7.3.3 Maximum soil-bearing pressure (on diagonal) should be calculated using the design aid for soil pressure for biaxial loaded footings as shown in *PIP STE03360*.

#### 4.7.4 Stability/Sliding

- 4.7.4.1 The minimum overturning “stability ratio” and the minimum factor of safety against sliding for service loads other than earthquake should be 1.5 in accordance with *PIP STC01015*.

- 4.7.4.2 The minimum overturning “stability ratio” and the minimum factor of safety against sliding for earthquake service loads shall be 1.0 in accordance with *PIP STC01015*. In addition, the minimum overturning “stability ratio” for the anchorage and foundations of skirt-supported vertical vessels classified as Occupancy Category IV in accordance with *ASCE/SEI 7-05*, Section 1.5.1 and Table 1-1, shall be 1.2 for the critical earthquake loads specified in *ASCE/SEI 7-05*, Section 15.7.10.5.

- 4.7.4.3 The stability ratio should be computed using the following formula:

$$S.R. = b/2e \quad (\text{Equation 15})$$

where:

$b$  = dimension of footing in the direction of the overturning moment, ft

$e$  = overturning moment at the base of the footing divided by the total vertical load, ft. The moment and loads should be factored in accordance with load combinations for uplift cases in *PIP STC01015*, Table 3 - *Loading Combinations - Allowable Stress Design (Service Loads)*.

#### 4.7.5 Reinforcement

##### 4.7.5.1 Standard Factored Strength Design

Reinforced concrete design using factored strength design loads should be in accordance with *ACI 318*. The critical section for moment and shear should be taken with respect to the face of a square with an area equivalent to that of the pedestal.

Moment should be checked at the face of the equivalent square. Shear, as a measure of diagonal tension, should be checked at the critical section specified in *ACI 318-05*, Section 11.1.3.1 (at a distance  $d$  from the face of the equivalent square). The moment and shear should be calculated for a 1-ft-wide strip as a simple cantilever from the edge of the equivalent square. Punching shear may need to be checked in some situations in accordance with *ACI 318*. The

resulting reinforcing steel should be placed continuously across the entire footing in a grid pattern, the minimum bottom reinforcement being #5 bars at 12 inches on-center, each way.

#### 4.7.5.2 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:

$$f_t = 5\phi(f_c')^{1/2} \quad (\text{Equation 16})$$

where:

$f_t$  = flexural strength of structural plain concrete, psi

$f_c'$  = 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.

See the following formulas for footing thicknesses that do not require top reinforcing steel:

For footings cast against soil:

$$t_{\text{reqd}} = t_{\text{eff}} + 2 \text{ inches} \quad (\text{Equation 17a})$$

For footings cast against a seal slab:

$$t_{\text{reqd}} = t_{\text{eff}} \quad (\text{Equation 17b})$$

with  $t_{\text{eff}}$  calculated as follows:

$$t_{\text{eff}} = (6M_u / f_t')^{1/2} \quad (\text{Equation 18})$$

where:

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

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

$M_u$  = factored moment caused by the weight of soil and concrete acting on a 1-inch strip in the footing at the face of the equivalent square pedestal, inch-pounds per inch, calculated using a load factor of 1.4

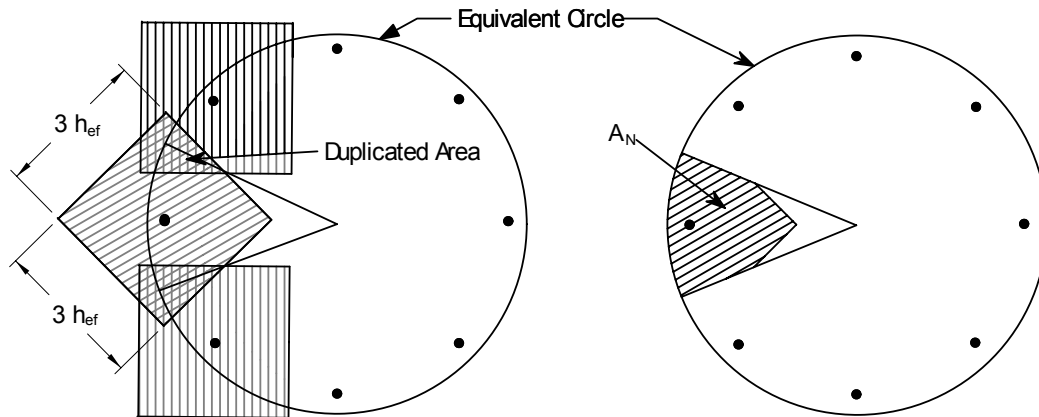
$f_t'$  = flexural strength of structural plain concrete, psi (from Equation 18)

## **APPENDIX:**

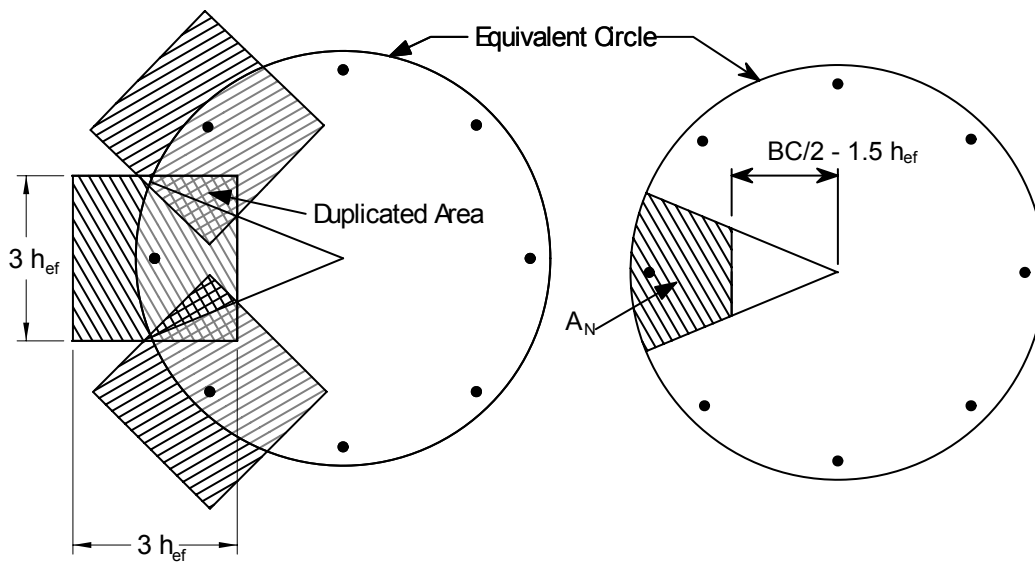
### **Figures, Tables, and Example**

**Figure A - Potential Concrete Failure Areas ( $A_N$ ) for Various Configurations**

Projected Concrete Failure Areas ( $A_N$ ) - Overlap:



Alternative:



(Figure A continues on next page.)

**Figure A (Continued) - Potential Concrete Failure Areas ( $A_N$ ) for Various Configurations**

Projected Concrete Failure Areas ( $A_N$ ) - Do Not Overlap:

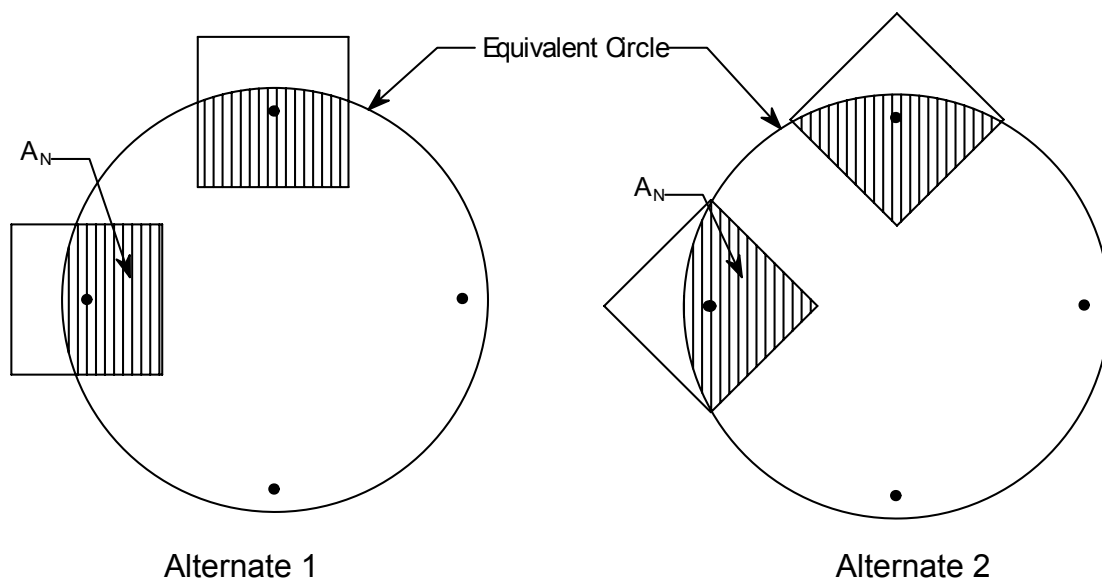
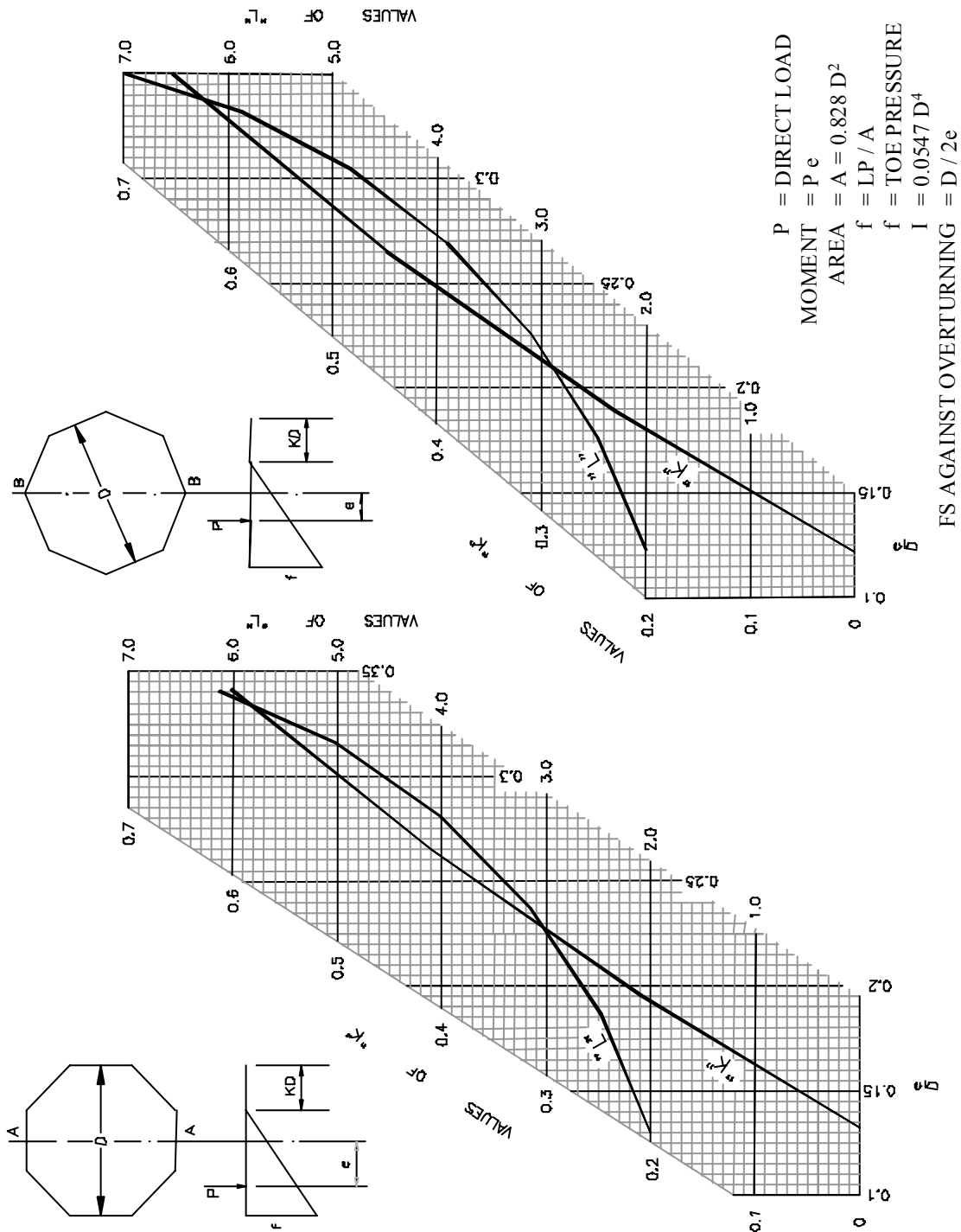
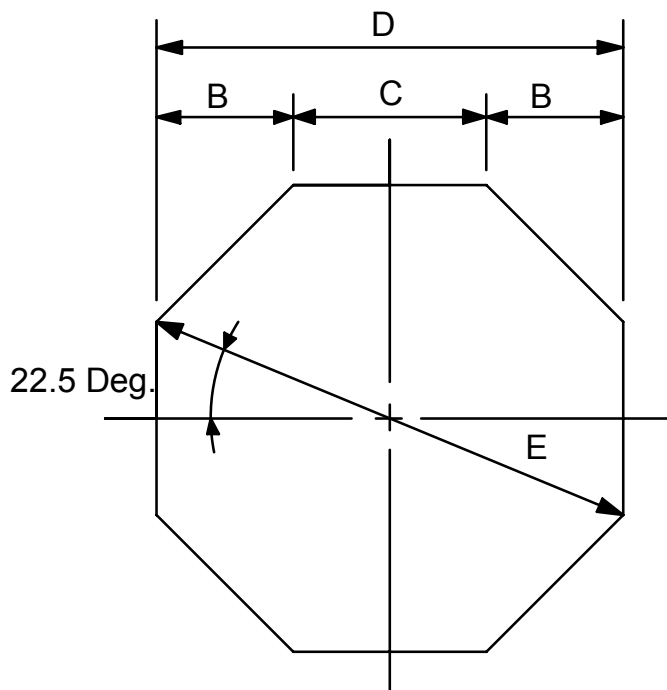




Figure B - Foundation Pressures for Octagon Bases



**Table 1 - Octagon Properties**



$A = \text{Area (SF)} = 0.8284272 D^2$   
 $B = C \times \sin 45^\circ = 0.2928932 D$   
 $C = \text{Length of Side} = 0.4142136 D$   
 $E = \text{Length of Diameter} = 1.0823922 D$   
 $Z_e = \text{Sec. Mod. Diameter} = 0.1011422 D^3$   
 $Z_d = \text{Sec. Mod. Flat} = Z_e/D$   
 $I = \text{Moment of Inertia} = Z_e E/2$

(Table 1 continues on following pages.)

(Table 1, continued)

<b>D</b> <b>ft – inch</b>	<b>D</b> <b>ft</b>	<b>C</b> <b>ft – inch</b>	<b>B</b> <b>ft – inch</b>	<b>A</b> <b>ft<sup>2</sup></b>	<b>Z<sub>s</sub></b> <b>ft<sup>3</sup></b>	<b>E</b> <b>ft</b>
6- 0-1/2	6.036	2- 6	1- 9-1/4	30.18	22.24	6.53
6- 2-7/8	6.237	2- 7	1- 9-15/16	32.23	24.54	6.75
6- 5-1/4	6.438	2- 8	1- 10-5/8	34.34	26.99	6.97
6- 7-5/8	6.639	2- 9	1- 11-5/8	36.51	29.60	7.19
6- 10-1/8	6.840	2- 10	2- 0-1/16	38.76	32.37	7.40
7- 0-1/2	7.042	2- 11	2- 0-3/4	41.08	35.32	7.62
7- 2-7/8	7.243	3- 0	2- 1-7/16	43.46	38.43	7.84
7- 5-3/8	7.444	3- 1	2- 2-3/16	45.91	41.72	8.06
7- 7-3/4	7.645	3- 2	2- 2-7/8	48.42	45.19	8.27
7- 10-1/8	7.846	3- 3	2- 3-9/16	51.00	48.85	8.49
8- 0-5/8	8.047	3- 4	2- 4-5/16	53.64	52.70	8.71
8- 3	8.249	3- 5	2- 5	56.37	56.77	8.93
8- 5-3/8	8.450	3- 6	2- 5-11/16	59.15	61.02	9.15
8- 7-7/8	8.651	3- 7	2- 6-7/16	62.00	65.48	9.36
8- 10-1/4	8.852	3- 8	2- 7-1/8	64.91	70.15	9.58
9- 0-5/8	9.053	3- 9	2- 7-13/16	67.90	75.04	9.80
9- 3	9.250	3- 10	2- 8-1/2	70.88	80.05	10.01
9- 5-1/2	9.456	3- 11	2- 9-1/4	74.07	85.52	10.24
9- 7-7/8	9.657	4- 0	2- 9-5/16	77.26	91.09	10.45
9- 10-3/8	9.858	4- 1	2- 10-11/16	80.51	96.89	10.67
10- 0-3/4	10.059	4- 2	2- 11-3/8	83.82	102.94	10.89
10- 3-1/8	10.260	4- 3	3- 0-1/16	87.21	109.24	11.10
10- 5-1/2	10.462	4- 4	3- 0-3/4	90.67	115.82	11.32
10- 8	10.663	4- 5	3- 1-1/2	94.19	122.62	11.54
10- 10-3/8	10.864	4- 6	3- 2-3/16	97.78	129.69	11.76
11- 0-3/4	11.065	4- 7	3- 2-7/8	101.43	137.02	11.97
11- 3-1/4	11.266	4- 8	3- 3-5/8	105.15	144.62	12.19
11- 5-5/8	11.468	4- 9	3- 4-15/16	108.95	152.54	12.41
11- 8	11.666	4- 10	3- 5	112.80	160.71	12.62
11- 10-3/8	11.870	4- 11	3- 5-11/16	116.72	169.15	12.85
12- 0-7/8	12.071	5- 0	3- 6-7/16	120.71	177.89	13.06
12- 3-1/4	12.272	5- 1	3- 7-1/8	124.76	186.93	13.28
12- 5-5/8	12.474	5- 2	3- 7-13/16	128.90	196.31	13.49
12- 8-1/8	12.675	5- 3	3- 8-9/16	133.09	205.96	13.72
12- 10-1/2	12.876	5- 4	3- 9-1/4	137.35	215.91	13.93
13- 0-7/8	13.077	5- 5	3- 9-15/16	141.67	226.18	14.15
13- 3-3/8	13.278	5- 6	3- 10-11/16	146.06	236.77	14.37
13- 5-3/4	13.479	5- 7	3- 11-3/8	150.51	247.69	14.58
13- 8-1/8	13.681	5- 8	4- 0-1/16	155.06	258.99	14.80
13- 10-5/8	13.882	5- 9	4- 0-13/16	159.65	270.58	15.03
14- 1	14.083	5- 10	4- 1-1/2	164.30	282.50	15.24
14- 3-3/8	14.284	5- 11	4- 2-3/16	169.03	294.77	15.46
14- 5-7/8	14.485	6- 0	4- 2-15/16	173.62	307.39	15.68
14- 8-1/4	14.686	6- 1	4- 3-5/8	178.67	320.36	15.90
14- 10-5/8	14.888	6- 2	4- 4-5/16	183.62	333.77	16.11

<b>D</b> <b>ft – inch</b>	<b>D</b> <b>ft</b>	<b>C</b> <b>ft – inch</b>	<b>B</b> <b>ft – inch</b>	<b>A</b> <b>ft<sup>2</sup></b>	<b>Z<sub>s</sub></b> <b>ft<sup>3</sup></b>	<b>E</b> <b>ft</b>
15- 1-1/8	15.088	6- 3	4- 5-1/16	188.59	347.40	16.33
15- 3-1/2	15.290	6- 4	4- 5-3/4	193.67	361.54	16.55
15- 5-7/8	15.491	6- 5	4- 6-7/16	195.80	375.99	16.77
15- 8-1/4	15.692	6- 6	4- 7-1/8	203.99	390.81	16.98
15- 10-3/4	15.893	6- 7	4- 7-7/8	209.25	406.02	17.20
16- 1-1/8	16.095	6- 8	4- 8-9/16	214.60	421.70	17.42
16- 3-5/8	16.296	6- 9	4- 9-5/16	220.00	437.70	17.64
16- 6	16.497	6- 10	4- 10	225.46	454.10	17.85
16- 8-3/8	16.698	6- 11	4- 10-11/16	230.98	470.90	18.07
16- 10-3/4	16.899	7- 0	4- 11-3/8	236.58	488.11	18.23
17- 1-1/4	17.101	7- 1	5- 0-1/8	242.27	505.82	18.51
17- 3-5/8	17.302	7- 2	5- 0-13/16	248.00	523.87	18.72
17- 6	17.503	7- 3	5- 1-1/2	253.79	542.34	18.95
17- 8-1/2	17.704	7- 4	5- 2-1/4	259.66	561.24	19.16
17- 10-7/8	17.905	7- 5	5- 2-15/16	265.58	580.57	19.38
18- 1-1/4	18.107	7- 6	5- 3-5/8	271.61	600.44	19.60
18- 3-3/4	18.308	7- 7	5- 4-3/8	277.67	620.66	19.82
18- 6-1/8	18.509	7- 8	5- 5-1/16	283.81	641.33	20.03
18- 8-1/2	18.710	7- 9	5- 5-3/4	290.00	662.45	20.25
18- 10-7/8	18.911	7- 10	5- 6-7/16	296.27	683.03	20.47
19- 1-3/8	19.113	7- 11	5- 7-3/16	302.63	706.19	20.69
19- 3-3/4	19.313	8- 0	5- 7-7/8	309.00	728.59	20.90
19- 6-1/8	19.515	8- 1	5- 8-9/16	315.49	751.69	21.12
19- 8-5/8	19.716	8- 2	5- 9-5/16	322.03	775.16	21.34
19- 11	19.917	8- 3	5- 10	328.63	799.11	21.56
20- 1-3/8	20.118	8- 4	5- 10-11/16	335.30	823.54	21.78
20- 3-7/8	20.320	8- 5	5- 11-7/16	342.06	848.60	21.99
20- 6-1/4	20.521	8- 6	6- 0-1/8	348.86	874.03	22.21
20- 8-3/4	20.722	8- 7	6- 0-7/8	355.72	899.97	22.43
20- 11-1/8	20.923	8- 8	6- 1-9/16	362.66	926.41	22.65
21- 1-1/2	21.124	8- 9	6- 2-1/4	369.66	953.37	22.86
21- 3-7/8	21.325	8- 10	6- 2-15/16	376.73	980.84	23.05
21- 6-1/4	21.527	8- 11	6- 3-5/8	383.90	1008.98	23.30
21- 8-3/4	21.728	9- 0	6- 4-3/8	391.10	1037.50	23.52
21- 11-1/4	21.929	9- 1	6- 5-1/16	398.37	1067.57	23.74
22- 1-5/8	22.130	9- 2	6- 5-13/16	405.70	1096.22	23.95
22- 4	22.331	9- 3	6- 6-1/2	413.17	1126.31	24.17
22- 6-3/8	22.533	9- 4	6- 7-3/16	420.62	1157.15	24.39
22- 8-7/8	22.734	9- 5	6- 7-15/16	428.16	1188.39	24.61
22- 11-1/4	22.935	9- 6	6- 8-5/8	435.76	1220.19	24.83
23- 1-5/8	23.136	9- 7	6- 9-5/16	443.44	1252.56	25.04
23- 4	23.337	9- 8	6- 10	451.17	1285.49	25.26
23- 6-1/2	23.539	9- 9	6- 10-3/4	459.02	1319.16	25.48
23- 8-7/8	23.739	9- 10	6- 11-7/16	466.85	1353.07	25.72
23- 11-3/8	23.941	9- 11	7- 0-3/16	474.83	1387.90	25.91
24- 1-3/4	24.142	10- 0	7- 0-7/8	482.84	1423.15	26.14
24- 4-1/8	24.343	10- 1	7- 1-9/16	490.91	1459.00	26.35

<b>D</b> <b>ft – inch</b>	<b>D</b> <b>ft</b>	<b>C</b> <b>ft – inch</b>	<b>B</b> <b>ft – inch</b>	<b>A</b> <b>ft<sup>2</sup></b>	<b>Z<sub>s</sub></b> <b>ft<sup>3</sup></b>	<b>E</b> <b>ft</b>
24- 6-1/2	24.545	10- 2	7- 2-1/4	499.09	1495.62	26.56
24- 9	24.746	10- 3	7- 3	507.30	1532.67	26.78
24- 11-3/8	24.947	10- 4	7- 3-11/16	515.57	1570.32	27.00
25- 1-3/4	25.148	10- 5	7- 4-3/8	523.92	1608.58	27.22
25- 4-1/4	25.349	10- 6	7- 5-1/8	532.32	1647.46	27.44
25- 6-5/8	25.550	10- 7	7- 5-13/16	540.80	1686.90	27.66
25- 9	25.752	10- 8	7- 6-1/2	549.38	1727.29	27.87
25- 11-1/2	25.953	10- 9	7- 7-1/4	557.99	1768.05	28.09
26- 1-7/8	26.154	10- 10	7- 7-15/16	566.67	1809.45	28.31
26- 4-3/8	26.355	10- 11	7- 8-11/16	575.41	1851.49	28.53
26- 6-5/8	26.556	11- 0	7- 9-5/16	584.22	1894.18	28.74
26- 9-1/8	26.757	11- 1	7- 10-3/16	593.10	1937.51	28.96
26- 11-1/2	26.959	11- 2	7- 10-3/4	602.09	1981.73	29.18
27- 1-7/8	27.160	11- 3	7- 11-7/16	611.10	2026.38	29.40
27- 4-3/8	27.361	11- 4	8- 0-3/16	620.18	2071.71	29.62
27- 6-3/4	27.562	11- 5	8- 0-7/8	629.33	2117.70	29.83
27- 9-1/8	27.763	11- 6	8- 1-9/16	638.54	2164.37	30.05
27- 11-5/8	27.965	11- 7	8- 2-5/16	647.86	2217.96	30.27
28- 2	28.165	11- 8	8- 3	657.21	2260.00	30.49
28- 4-1/2	28.376	11- 9	8- 3-3/4	667.05	2310.93	30.71
28- 6-3/4	28.568	11- 10	8- 4-3/8	675.86	2356.91	30.92
28- 9-1/4	28.769	11- 11	8- 3-1/8	685.65	2408.29	31.14
28- 11-5/8	28.970	12- 0	8- 5-13/16	695.31	2459.36	31.36
29- 2-1/8	29.171	12- 1	8- 6-9/16	704.95	2510.65	31.57
29- 4-1/2	29.373	12- 2	8- 7-1/4	714.74	2563.16	31.79
29- 6-7/8	29.574	12- 3	8- 7-15/16	724.56	2816.15	32.01
29- 9-1/4	29.775	12- 4	8- 8-5/8	734.44	2669.85	32.23
29- 11-3/4	29.977	12- 5	8- 9-3/8	744.44	2724.56	32.45
30- 2-1/8	30.178	12- 6	8- 10-1/16	754.45	2779.73	32.66
30- 4-1/2	30.379	12- 7	8- 10-3/4	764.54	2835.65	32.88
30- 7	30.580	12- 8	8- 11-1/2	774.69	2892.30	33.10
30- 9-3/8	30.781	12- 9	9- 0-3/16	784.91	2949.72	33.32
30- 11-3/4	30.982	12- 10	9- 0-7/8	795.19	3007.88	33.53
31- 2-1/4	31.184	12- 11	9- 1-5/8	805.60	3067.10	33.75
31- 4-5/8	31.385	13- 0	9- 2-5/16	816.01	3126.79	33.99
31- 7	31.586	13- 1	9- 3	826.45	3186.95	34.19
31- 9-1/2	31.787	13- 2	9- 3-3/4	837.05	3248.45	34.41
31- 11-7/8	31.988	13- 3	9- 4-7/16	847.67	3310.50	34.62
32- 2-1/4	32.189	13- 4	9- 5-1/8	858.35	3373.30	34.84
32- 4-3/4	32.391	13- 5	9- 5-7/8	869.17	3437.21	35.06
32- 7-1/8	32.592	13- 6	9- 6-9/16	879.98	3501.59	35.28
32- 9-1/2	32.793	13- 7	9- 7-1/4	890.87	3566.78	35.49
33- 0	32.994	13- 8	9- 8	901.82	3632.74	35.71
33- 2-3/8	33.195	13- 9	9- 8-11/16	912.85	3699.56	35.93
32- 4-3/4	33.397	13- 10	9- 9-3/8	923.99	3767.51	36.15
33- 7-1/8	33.597	13- 11	9- 10-1/16	935.09	3835.61	36.37
33- 9-5/8	33.799	14- 0	9-10-13/16	946.37	3905.21	36.58

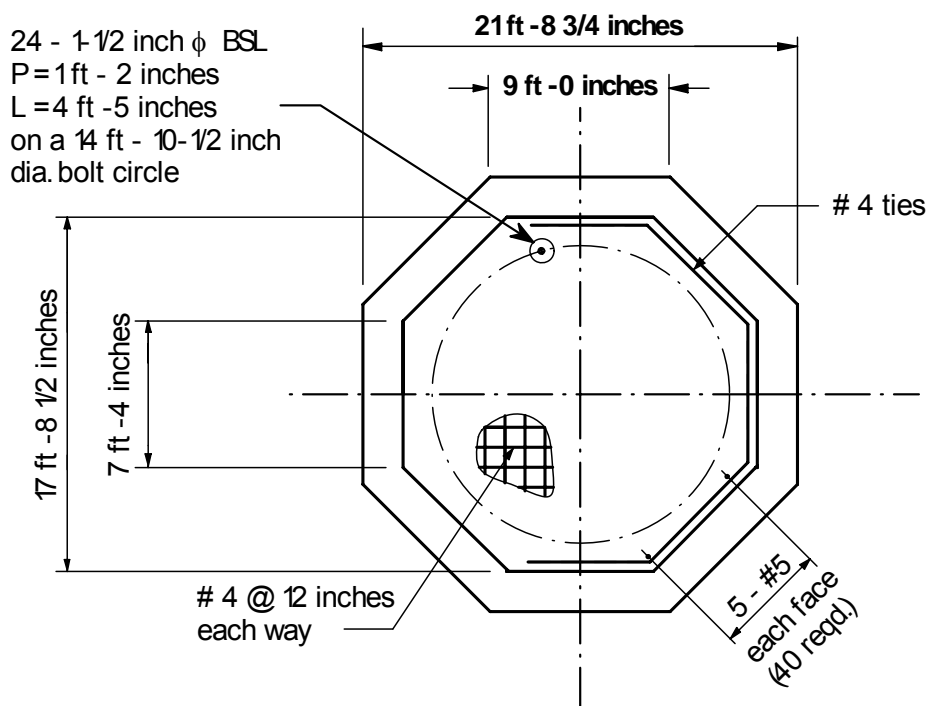
<b>D</b> <b>ft – inch</b>	<b>D</b> <b>ft</b>	<b>C</b> <b>ft – inch</b>	<b>B</b> <b>ft – inch</b>	<b>A</b> <b>ft<sup>2</sup></b>	<b>Z<sub>s</sub></b> <b>ft<sup>3</sup></b>	<b>E</b> <b>ft</b>
34- 0	34.009	14- 1	9- 11-1/2	958.17	3978.45	36.81
34- 2-1/2	34.201	14- 2	10- 0-1/4	969.02	4046.21	37.02
34- 4-7/8	34.403	14- 3	10- 0-15/16	980.50	4118.33	37.24
34- 7-1/4	34.603	14- 4	10- 1-5/8	991.97	4190.81	37.45
34- 9-3/4	34.805	14- 5	10- 2-3/8	1003.55	4264.39	37.67
35- 0-1/8	35.006	14- 6	10- 3-1/16	1015.18	4338.74	37.89
35- 2-1/2	35.207	14- 7	10- 3-3/4	1026.86	4413.89	38.11
35- 5	35.409	14- 8	10- 4-1/2	1038.65	4490.11	38.33
35- 7-3/8	35.610	14- 9	10- 5-3/16	1050.51	4567.18	38.54
35- 9-3/4	35.811	14- 10	10- 5-7/8	1062.38	4644.86	38.76
36- 0-3/4	36.012	14- 11	10- 6-5/8	1074.36	4723.61	38.98
36- 2-5/8	36.213	15- 0	10- 7-5/16	1086.40	4803.23	39.20
36- 5	36.414	15- 1	10- 8	1098.48	4883.57	39.41
36- 7-1/2	36.616	15- 2	10- 8-3/4	1110.68	4965.15	39.63
36-9-7/8	36.817	15- 3	10- 9-7/16	1122.93	5047.51	39.85
37- 0-1/4	37.018	15- 4	10- 10-1/8	1135.21	5130.58	40.07
37- 2-3/4	37.219	15- 5	10- 10-7/8	1147.58	5214.67	40.29
37- 5-1/8	37.420	15- 6	10- 11-9/16	1160.03	5299.74	40.50
37- 7-1/2	37.621	15- 7	11- 0-1/4	1172.51	5385.47	40.72
37- 9-7/8	37.822	15- 8	11- 0-15/16	1185.12	5472.58	40.94
38- 0-3/8	38.024	15- 9	11- 1-11/16	1197.76	5560.40	41.16
38- 2-3/4	38.225	15- 10	11- 2-3/8	1210.45	5649.03	41.37
38- 5-1/8	38.426	15- 11	11- 3-1/16	1223.22	5738.63	41.59
38- 7-5/8	38.627	16- 0	11- 3-13/16	1236.08	5829.34	41.81
38- 10	38.829	16- 1	11- 4-1/2	1249.01	5921.08	42.03
39- 0-3/8	39.030	16- 2	11- 5-3/16	1261.97	6013.45	42.25
39- 2-7/8	39.231	16- 3	11- 5-15/16	1275.01	6106.90	42.46
39- 5-1/4	39.432	16- 4	11- 6-5/8	1288.11	6201.28	42.68
39- 7-5/8	39.633	16- 5	11- 7-15/16	1301.27	6296.56	42.90
39- 10-1/8	39.835	16- 6	11- 8-1/16	1314.54	6393.09	43.12

**Table 2 - Foundation Pressures for Octagon Bases  
(Large Eccentricities - Load Perpendicular to Face)**

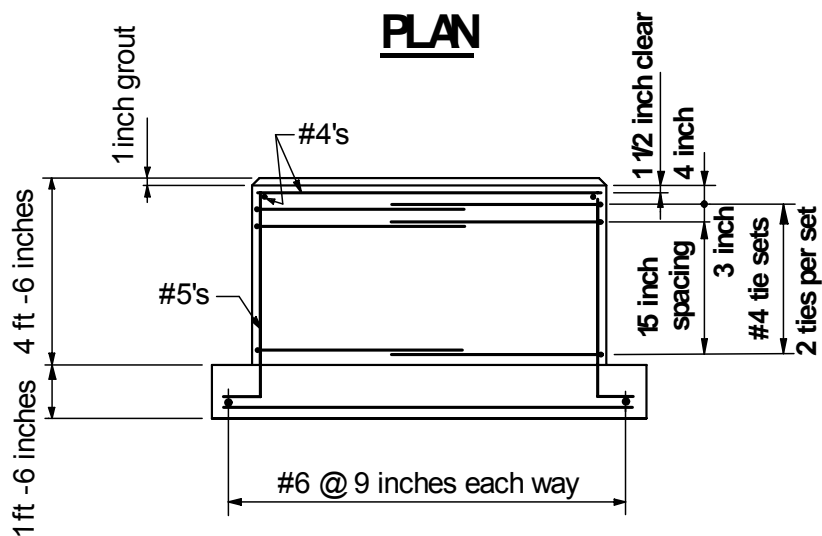
e/D	K	L	e/D	K	L	e/D	K	L
0.300	0.4935	4.503	0.370	0.6676	7.844	0.440	0.8369	19.432
0.305	0.5065	4.656	0.375	0.6794	8.233	0.445	0.8496	21.421
0.310	0.5195	4.819	0.380	0.6912	8.654	0.450	0.8625	23.812
0.315	0.5323	4.991	0.385	0.7030	9.113	0.455	0.8754	26.742
0.320	0.5450	5.174	0.390	0.7149	9.615	0.460	0.8885	30.411
0.325	0.5577	5.369	0.395	0.7267	10.167	0.465	0.9017	35.138
0.330	0.5703	5.576	0.400	0.7387	10.775	0.470	0.9151	41.452
0.335	0.5828	5.797	0.405	0.7507	11.450	0.475	0.9287	50.304
0.340	0.5951	6.032	0.410	0.7628	12.203	0.480	0.9424	63.601
0.345	0.6074	6.284	0.415	0.7749	13.046	0.485	0.9564	85.785
0.350	0.6196	6.553	0.420	0.7872	13.998	0.490	0.9707	130.192
0.355	0.6317	6.842	0.425	0.7995	15.080	0.495	0.9852	263.487
0.360	0.6438	7.152	0.430	0.8119	16.320	0.500	See Note	See Note
0.365	0.6557	7.485	0.435	0.8244	17.754	> 0.500	See Note	See Note

Note: For e/D values greater or equal to 0.500, assume soil bearing as a line load with a length equal to the face dimension of the octagon footing applied at a distance of e from the centerline of the footing.

**Example - Vertical Vessel Foundation Design**

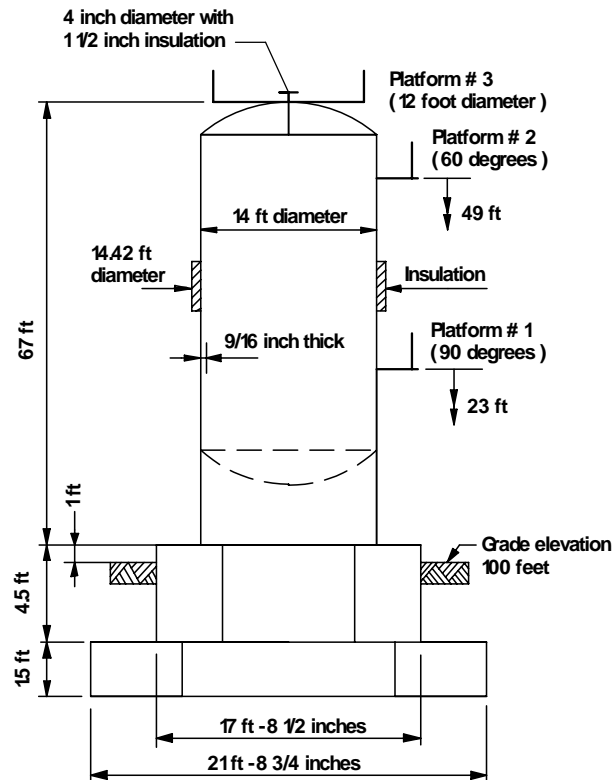


**PLAN**



**SECTION**





## DESIGN DATA

### Wind Load

In accordance with *ASCE/SEI 7-05*  
( $V = 115$  mph)

### Vessel Data

Empty wt. ( $D_e$ ) = 170.3 kip

Oper. wt. ( $D_o$ ) = 345.2 kip

Test wt. ( $D_t$ ) = 624.1 kip

### Structural Data

Allowable net soil bearing pressure at a depth of 5 feet for transient load combinations (i.e. load combinations that include wind loads) = 3.25 ksf

$\gamma = 110$  pcf

$f'_c = 4,000$  psi

$f_y = 60,000$  psi

### From Wind Load Analysis

(Nominal loads)

$V = 44.75$  kip

$M = 1,902$  kip-ft (at top of grout)

### Anchor Bolts

24, 1-1/2-inch  $\varnothing$  type

*ASTM F1554*, Grade 36, with a 4-inch  $\varnothing$  x 15-inch-long sleeve and 1-ft - 2-inch projection on a 14-ft - 10-1/2-inch  $\varnothing$  bolt circle (nonpretensioned)

## WIND/EARTHQUAKE LOAD VERIFICATION

*Note:* Wind/earthquake load calculations are not shown because they are beyond the scope of this Practice.

## PEDESTAL DESIGN

### Pedestal Dimensions and Weight

$$BC + 9 \text{ inches} = 178.5 \text{ inches} + 9 \text{ inches} = 187.5 \text{ inches} \quad (\text{Equation 1a, this Practice})$$

$$BC + 8(BD) = 178.5 \text{ inches} + 8(1.5 \text{ inches}) = 190.5 \text{ inches} \quad (\text{Equation 1b, this Practice})$$

$$BC + SD + 9 \text{ inches} - BD = 178.5 \text{ inches} + 4 \text{ inches} + 9 \text{ inches} - 1.5 \text{ inches} = 190.0 \text{ inches} \quad (\text{Equation 1d, this Practice})$$

$$BC + SD + 7(BD) = 178.5 + 4 + 7(1.5 \text{ inches}) = 193 \text{ inches} \quad \blacktriangleleft \text{Controls} = 16.083 \text{ ft} \quad (\text{Equation 1e, this Practice})$$

- Use 16-ft - 1-1/8-inch octagon.

*Note:* Pedestal "diameter" had to be increased to 17-ft - 8-1/2-inches to provide a sufficiently large projected concrete failure area to resist the tensile load in the anchor bolts. (See "Projected Concrete Failure Area" under "Anchor Bolt Check," as follows.) Alternatively, additional reinforcing steel may be used to transfer anchor bolt forces to concrete.

### Pedestal Reinforcement

$$\text{Pedestal area} = 259.7 \text{ ft}^2 \quad (\text{Table 1, this Practice})$$

$$\text{Pedestal weight } (D_p) = (259.7 \text{ ft}^2)(4.5 \text{ ft})(0.15 \text{ kcf}) = 175.3 \text{ kip}$$

$$M_{ped} = \text{O.T.M. at pedestal base} = (1,902 \text{ kip-ft}) + (4.5 \text{ ft})(44.75 \text{ kip}) = 2,104 \text{ kip-ft}$$

$$Mu_{ped} = 1.6M_{ped} = 1.6(2,104 \text{ kip-ft}) = 3,366 \text{ kip-ft}$$

(Load Factors should be in accordance with *PIP STC01015*, Table 4, Load Comb. 4)

$$N_d = \text{number of dowels} = \text{assume } 40$$

$$DC = (17.71\text{-ft pedestal}) - (\text{say } 0.5 \text{ ft}) = 17.21 \text{ ft}$$

$$D_e + D_p = \text{empty weight of vessel} + \text{pedestal weight} = 170.3 \text{ kip} + 175.3 \text{ kip} = 345.6 \text{ kip}$$

$$F_u = 4(Mu_{ped}) / [(N_d)(DC)] - 0.9(D_e + D_p) / N_d = 4(3,366 \text{ kip-ft}) / [(40)(17.21 \text{ ft})] - 0.9(345.6 \text{ kip}) / 40$$

$$= 11.78 \text{ kip} \quad (\text{Equation 2, this Practice})$$

$$A_{s_{req'd}} = F_u / \phi f_y = (11.78 \text{ kip}) / (0.9)(60 \text{ ksi}) = 0.22 \text{ inch}^2 \quad (\text{Equation 3, this Practice})$$

- Use 40 #5 bars ( $A_s = 0.31 \text{ inch}^2$ ) with #4 ties at 15 inches c/c (minimum reinforcement controls)

### Anchor Bolt Check

#### Maximum Tension on Anchor Bolt:

$$N_u = 4M_u / [(N_b)(BC)] - 0.9(D_e)/N_b \quad (\text{Equation 4, this Practice})$$

$$N_u = 4[(1.6)(1,902 \text{ kip-ft})] / [(24)(14.88 \text{ ft})] - 0.9(170.3 \text{ kips}) / 24 = 27.7 \text{ kips}$$

(Load factors should be in accordance with *PIP STC01015*, Table 4, Load Comb. 4)

#### Maximum Shear on Anchor Bolt:

$$V_u \text{ at base top of grout} = 1.6(44.75 \text{ kip}) = 71.6 \text{ kips}$$

Check whether shear load can be taken by friction between base of vessel and top of grout.

$$P_u = M_u / LA + 0.9(D_e) / 2 \quad (\text{Equation 5, this Practice})$$

Conservatively, take LA as 2/3 of BC diameter = (2/3)(14.875 ft) = 9.92 ft

$$P_u = 1.6(1902 \text{ kip-ft}) / (9.92 \text{ ft}) + 0.9(170.3 \text{ kips}) / 2 = 307 \text{ kips} + 77 \text{ kips} = 384 \text{ kips}$$

$$V_f = \mu P_u = (0.55)(384 \text{ kips}) = 211 \text{ kips} \quad (\text{Equation 6, this Practice})$$

$$\phi V_f = (0.75)(211 \text{ kips}) = 158 \text{ kips} > 71.6 \text{ kips} \quad (\text{Equation 7, this Practice})$$

- Therefore, the anchor bolts are not required to resist shear.

#### Projected Concrete Failure Area:

*Note:* Several iterations were required to determine that  $D = 16 \text{ ft} - 1\text{-}1/8 \text{ inches}$  would not provide enough projected concrete failure area to resist the maximum tensile load  $N_u = 27.7 \text{ kips}$ , regardless of what embedment depth,  $h_{ef}$ , was used. To save space, these trial calculations are not shown here. Reinforcing steel either should be added to transfer the tensile load from the anchor bolts to the pedestal or the pedestal “diameter” should be increased. This second alternative is shown here.

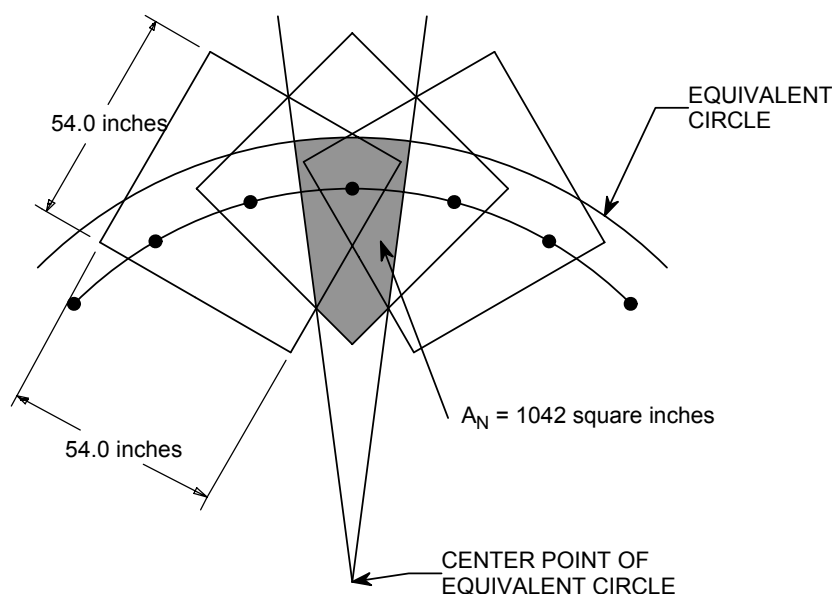
Try increasing  $D$  to  $17 \text{ ft} - 8\text{-}1/2 \text{ inches}$  (17.704 ft),

$$h_{ef} = 18 \text{ inches} = 1.50 \text{ ft} \quad (\text{Table 1, PIP STE05121})$$

$$1.5 h_{ef} = 2.25 \text{ ft}$$

$$\text{Equivalent circle diameter, } D_{EQ} = (1.027)(17.704 \text{ ft}) = 18.18 \text{ ft}$$

Determine  $A_N$  graphically:



Using the PIP *Anchor Bolt Design Spreadsheet* (available to PIP Member Companies only) or *ACI 318-05* Appendix D, the following was determined:

$$\phi N_n = 33.2 \text{ kips} > N_u = 27.7 \text{ kips} \quad \text{O.K.}$$

Bolt length required (L) = projection +  $h_{ef}$  + projection at bottom (P1) from Table 1 in *PIP STE05121*:

$$L (\text{min}) = (1 \text{ ft} - 2 \text{ inches}) + (1 \text{ ft} - 6 \text{ inches}) + (2 \text{ inches}) = 2 \text{ ft} - 10 \text{ inches}$$

Use standard length bolts in accordance with *PIP STF05121*. Determine whether ASL or BSL bolts are required. (See *PIP STF05121* for definition of ASL and BSL.)

For 1-1/2-inch-diameter bolts, ASL = 2 ft - 8 inches, BSL = 4 ft - 5 inches

Required bolt length is too long for ASL bolts, use BSL bolts. BSL bolt embedment including the following:

$$P1 = (4 \text{ ft} - 5 \text{ inches}) - (1 \text{ ft} - 2 \text{ inches}) = 3 \text{ ft} - 3 \text{ inches} < \text{pedestal depth} \\ (4 \text{ ft} - 6 \text{ inches})$$

Therefore, anchor bolts will not extend into the bottom mat. O.K.

- Use 24 - 1-1/2-inch-diameter BSL anchor bolts.
- Revise “diameter” of pedestal octagon to 17 ft - 8-1/2 inches (face to face).

## FOOTING DESIGN

Size footing: Because of constructability considerations, a square foundation may be more economical; however an octagon-shaped foundation is shown here to illustrate the procedure for this type of foundation.

### Select a Trial Octagon Size:

$$M_{ftg} = \text{O.T.M. at footing base} = (1,902 \text{ kip-ft}) + (6.0 \text{ ft})(44.75 \text{ kip}) \\ = 2,171 \text{ kip-ft}$$

$$SB = \text{allowable gross soil bearing} = (3.25 \text{ ksf}) + (5 \text{ ft})(0.11 \text{ kcf}) = 3.80 \text{ ksf}$$

$$\text{Trial diameter} = (2.6)(M_{ftg}/SB)^{1/3} = (2.6)[(2,171 \text{ kip-ft})/(3.80 \text{ ksf})]^{1/3} = 21.57 \text{ ft} \\ \text{(Equation 8, this Practice)}$$

$$\text{Try a 21-ft - 8-3/4-inch octagon. Area} = 391.1 \text{ ft}^2 \quad \text{(Table 1, this Practice)}$$

### Check Required Thickness for Pedestal Reinforcing Embedment:

For #5 hooked bar,

$$l_{dh} = [(0.02\Psi_e\lambda f_y)/\sqrt{f'_c}](d_b) \quad \text{(ACI 318-05, Section 12.5.2)}$$

$$= [(0.02)(1.0)(1.0)(60,000 \text{ psi})/\sqrt{4,000 \text{ psi}}](0.625 \text{ inches}) = 11.9 \text{ inches}$$

$$A_{s\text{req'd}} / A_{s\text{prov}} = (0.22 \text{ inch}^2)/(0.31 \text{ inch}^2) = 0.71$$

$$\text{Treq'd} = (3 \text{ inch clear}) + (2 \text{ layers})(0.75 \text{ inch bar}) + (0.71)(0.7)(11.9 \text{ inches}) \\ = 10.4 \text{ inches}$$

$$T_{min} = 12 \text{ inches} \quad \text{(Section 4.7.1, this Practice)}$$

Try footing thickness = 18 inches

### Footing Weights

$$\text{Net weight of pedestal} = (259.7 \text{ ft}^2)[(4.5 \text{ ft})(0.15 \text{ kcf}) - (3.5 \text{ ft})(0.11 \text{ kcf})] \\ = 75.3 \text{ kip}$$

$$\text{Weight of footing + soil} = (391.1 \text{ ft}^2)[(1.5 \text{ ft})(0.15 \text{ kcf}) + (3.5 \text{ ft})(0.11 \text{ kcf})] \\ = 238.6 \text{ kip}$$

$$\text{Total } (D_s) = 75.3 \text{ kip} + 238.6 \text{ kip} = 313.9 \text{ kip}$$

$$P_e = D_e + D_s = 170.3 \text{ kip} + 313.9 \text{ kip} = 484.2 \text{ kip}$$

$$P_o = D_o + D_s = 345.2 \text{ kip} + 313.9 \text{ kip} = 659.1 \text{ kip}$$

$$P_t = D_t + D_s = 624.1 \text{ kip} + 313.9 \text{ kip} = 938.0 \text{ kip}$$

### Check Soil Bearing and Stability

Empty + wind (PIP STC01015, Table 3, Load Comb. 3):

$$P = P_e = 484.2 \text{ kip} \quad M_{ftg} = 2,171 \text{ kip-ft}$$

$$e = M_{ftg}/P = (2,171 \text{ kip-ft})/(484.2 \text{ kip}) = 4.48 \text{ ft}$$

Stability ratio =  $b/2e = (21.73 \text{ ft})/[2(4.48 \text{ ft})] = 2.43 > 1.5 \therefore$  O.K.  
(Equation 15, this Practice)

$e/D = (4.48 \text{ ft})/(21.73 \text{ ft}) = 0.206 > 0.122 \therefore L_{\text{diag}} = 2.85$   
(Figure B, this Practice)

$f = LP/A = (2.85)(484.2 \text{ kip})/(391.1 \text{ ft}^2) = 3.53 \text{ ksf} < 3.80 \text{ ksf} \therefore$  O.K.

Operating + wind (*PIP STC01015*, Table 3, Load Comb. 2):

$$P = P_o = 659.1 \text{ kip} \quad M_{\text{fig}} = 2,171 \text{ kip-ft}$$

$$e = M_{\text{fig}}/P = (2,171 \text{ kip-ft})/(659.1 \text{ kip}) = 3.29 \text{ ft}$$

$e/D = (3.29 \text{ ft})/(21.73 \text{ ft}) = 0.152 > 0.122 \therefore L_{\text{diag}} = 2.25$   
(Figure B, this Practice)

$f = LP/A = (2.25)(659.1 \text{ kip})/(391.1 \text{ ft}^2)$   
 $= 3.79 \text{ ksf} < 3.80 \text{ ksf} \therefore$  O.K. (controlling case) (Equation 11, this Practice)

Test + partial wind (*PIP STC01015*, Table 3, Load Comb. 6):

$$P = P_t = 938.0 \text{ kip}$$

Partial wind velocity = 68 mph

$$M_{\text{fig}} = (68 \text{ mph}/115 \text{ mph})^2(2,171 \text{ ft-kip}) = 759.1 \text{ ft-kip}$$

$$e = M_{\text{fig}}/P = (759.1 \text{ ft-kip})/(938.0 \text{ kip}) = 0.81 \text{ ft}$$

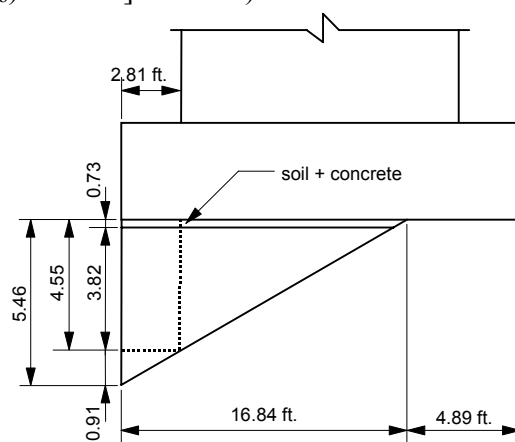
$$e/D = (0.81 \text{ ft})/(21.73 \text{ ft}) = 0.037 < 0.122$$

$f = P/A [1 + (8.19)(e/D)]$  (Equation 10a, this Practice)  
 $= [(938.0 \text{ kip})/(391.1 \text{ ft}^2)][1 + (8.19)(0.037)] = 3.13 \text{ ksf} < 3.80 \text{ ksf} \therefore$  O.K.

- Use 21-ft - 8-3/4-inch octagon.

### Bottom Reinforcement

Check operating + wind (in this instance, *PIP STC01015*, Table 4, Load Comb. 3,  $[1.2(D_s + D_o) + 1.6W]$  controls):



$$P_u = 1.2(659.1 \text{ kip}) = 790.9 \text{ kip}$$

$$M_u = 1.6(2,171 \text{ kip-ft}) = 3,474 \text{ kip-ft}$$

$$e = M_u/P_u = (3,474 \text{ kip-ft})/(790.9 \text{ kip}) = 4.39 \text{ ft}$$

$$e/D = (4.39 \text{ ft})/(21.73 \text{ ft}) = 0.202 > 0.132 \text{ (flat)}$$

$$L = 2.70 \text{ (flat)} \quad K = 0.225 \text{ (flat)}$$

(Figure B, this Practice)

$$KD = (0.225)(21.73 \text{ ft}) = 4.89 \text{ ft}$$

$$SB = LP/A = (2.70)(790.9 \text{ kip})/(391.1 \text{ ft}^2) = 5.46 \text{ ksf}$$

Find equivalent square for pedestal:

$$\text{side}^2 = 259.7 \text{ ft}^2 \quad \text{side} = 16.12\text{-ft} \quad \text{projection} = (21.73 \text{ ft} - 16.12 \text{ ft})/2 = 2.81 \text{ ft}$$

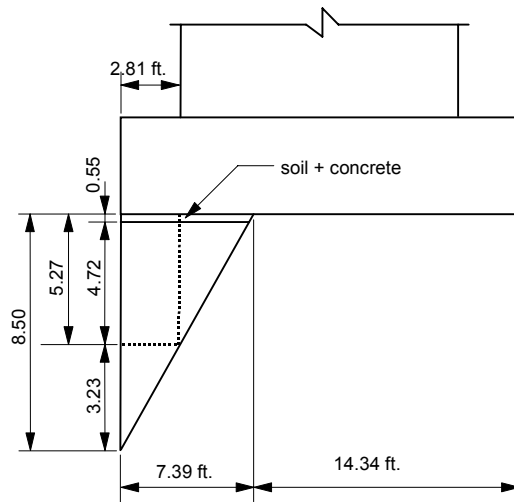
SB at face of equivalent square:

$$= 5.46 \text{ ksf}(16.84 \text{ ft} - 2.81 \text{ ft})/(16.84 \text{ ft}) = 4.55 \text{ ksf}$$

$$\text{Soil} + \text{concrete} = 1.2(238.6 \text{ kip})/(391.1 \text{ ft}^2) = 0.73 \text{ ksf}$$

$$M_u = (4.55 \text{ ksf} - 0.73 \text{ ksf})(2.81 \text{ ft})^2/2 + (5.46 \text{ ksf} - 4.55 \text{ ksf})(2.81 \text{ ft})^2/3 = 17.48 \text{ kip-ft}$$

Check empty + wind (in this instance, *PIP STC01015*, Table 4, Load Comb. 4,  
[0.9(D<sub>e</sub> + D<sub>s</sub>) + 1.6W] controls)



$$P_u = 0.9(484.2 \text{ kip}) = 435.8 \text{ kip}$$

$$M_u = 1.6(2,171 \text{ kip-ft}) = 3,474 \text{ kip-ft}$$

$$e = M_u/P_u = (3,474 \text{ kip-ft})/(435.8 \text{ kip}) = 7.97 \text{ ft}$$

$$e/D = (7.97 \text{ ft})/(21.73 \text{ ft}) = 0.367 > 0.132 \text{ (flat)}$$

$$L = 7.63 \text{ (flat)} \quad K = 0.660 \text{ (flat)}$$

(Table 2, this Practice)

$$KD = (0.660)(21.73 \text{ ft}) = 14.34 \text{ ft}$$

$$SB = LP_u/A = (7.63)(435.8 \text{ kip})/(391.1 \text{ ft}^2) = 8.50 \text{ ksf}$$

Find equivalent square for pedestal:

$$\text{side}^2 = 259.7 \text{ ft}^2$$

$$\text{side} = 16.12\text{-ft} \quad \text{projection} = (21.73 \text{ ft} - 16.12 \text{ ft})/2 = 2.81 \text{ ft}$$

SB at face of equivalent square:

$$= (8.50 \text{ ksf})(7.39 \text{ ft} - 2.81 \text{ ft})/(7.39 \text{ ft}) = 5.27 \text{ ksf}$$

$$\text{Soil} + \text{concrete} = 0.9(238.6 \text{ kip})/(391.1 \text{ ft}^2) = 0.55 \text{ ksf}$$

$$\begin{aligned} M_u &= (5.27 \text{ ksf} - 0.55 \text{ ksf})(2.81 \text{ ft})^2/2 + (8.50 \text{ ksf} - 5.27 \text{ ksf})(2.81 \text{ ft})^2/3 \\ &= 27.14 \text{ kip-ft} \quad \leftarrow \text{Controls} \end{aligned}$$

$$d = 18 \text{ inches} - 3 \text{ inches} - 1.125 \text{ inches} = 13.875 \text{ inches} = 1.16 \text{ ft}$$

$$F = bd^2/12,000 = (12 \text{ inches})(13.875 \text{ inches})^2/12,000 = 0.193$$

$$K_u = M_u/F = (27.14 \text{ kip-ft})/(0.193) = 140.6 \quad a_u = 4.390$$

$$A_s = M_u/(a_u d) = (27.14 \text{ kip-ft})/[(4.390)(13.875 \text{ inches})] = 0.45 \text{ inch}^2/\text{ft}$$

$$A_s \text{ min} = (0.0033)(12 \text{ inches})(13.875 \text{ inches}) = 0.55 \text{ inch}^2/\text{ft} \quad \leftarrow \text{Controls}$$

$$4/3 A_s = (4/3)(0.45 \text{ inch}^2/\text{ft}) = 0.60 \text{ inch}^2/\text{ft}$$

- Use #6 at 9 inches E.W. (bottom);  $A_s = 0.59 \text{ inch}^2/\text{ft}$ .

### Shear Check

#### Beam Shear – Empty + Wind Case:

SB (at distance d from face):

$$= (8.50 \text{ ksf})(7.39 \text{ ft} - 2.81 \text{ ft} + 1.16 \text{ ft})/(7.39 \text{ ft}) = 6.60 \text{ ksf}$$

$V_u$  (at distance d from face):

$$\begin{aligned} &= (6.60 \text{ ksf} - 0.55 \text{ ksf})(2.81 \text{ ft} - 1.16 \text{ ft}) + (8.50 \text{ ksf} - 6.60 \text{ ksf})(2.81 \text{ ft} - 1.16 \text{ ft})/2 \\ &= 9.98 \text{ kip/ft} + 1.57 \text{ kip/ft} \\ &= 11.55 \text{ kip/ft} \end{aligned}$$

$$v_u = (11.55 \text{ kip/ft})(1,000 \text{ lb/kip})/[(12 \text{ inches/ft})(13.875 \text{ inches})]$$

$$= 69.4 \text{ psi} < 2 \phi \sqrt{f'_c} = 94.9 \text{ psi} \quad \text{O.K.}$$

#### Punching Shear – Test Load Case:

$$P_u/A = 1.4(938.0 \text{ kip})/(391.1 \text{ ft}^2) = 3.36 \text{ ksf} \quad (\text{PIP STC01015, Table 4, Load Comb. 7})$$

$V_u$  (total at d/2 away from equivalent square)

$$\begin{aligned} &= [3.36 \text{ ksf} - (1.4/1.2)(0.73 \text{ ksf})][391.1 \text{ ft}^2 - (16.12 \text{ ft} + 1.16 \text{ ft})^2] \\ &= (2.51 \text{ ksf})(92.5 \text{ ft}^2) = 232 \text{ kip} \end{aligned}$$

$$b_o = 4(16.12 \text{ ft} + 1.16 \text{ ft}) = 69.1 \text{ ft}$$

$$\begin{aligned} v_u &= V_u/(db_o) = (232 \text{ kip})(1,000 \text{ lb/kip}) / [(13.875 \text{ inches})(69.1 \text{ ft})(12 \text{ inches/ft})] \\ &= 20 \text{ psi} \end{aligned}$$



$v_c$  (allowable) = the smaller of *ACI 318-05*, Equation 11-34 or 11-35

$$v_c \text{ (allowable)} = \phi (\alpha_s d/b_o + 2)(f'_c)^{1/2} \quad (\text{ACI 318-05, Equation 11-34})$$

$$= 0.75[(40)(1.16 \text{ ft})/(69.1 \text{ ft}) + 2](4,000 \text{ psi})^{1/2}$$

$$= 127 \text{ psi} > 20 \text{ psi} \quad \text{O.K.}$$

$$v_c \text{ (allowable)} = \phi (4)(f'_c)^{1/2} \quad (\text{ACI 318-05, Equation 11-35})$$

$$= 0.75(4)(4,000 \text{ psi})^{1/2}$$

$$= 190 \text{ psi} > 20 \text{ psi} \quad \text{O.K.}$$

### Top Reinforcement

Check to see if concrete can take weight of concrete plus soil above footing without top reinforcement. Use load factor of 1.4.

$$M_u = (1.4/1.2)(0.73 \text{ ksf})(2.81 \text{ ft})^2/2 = 3.36 \text{ kip-ft} = 3,360 \text{ inch-lb/inch}$$

$$f'_t = 5\phi(f'_c)^{1/2} = 5(0.55)(4,000 \text{ psi})^{1/2} = 173.9 \text{ psi} \quad (\text{Equation 16, this Practice})$$

$$t_{\text{reqd}} = t_{\text{eff}} + 2 \text{ inches}$$

$$= (6M_u/f'_t)^{1/2} + 2 \text{ inches}$$

$$= [6(3,360 \text{ inch-lb/inch})/(173.9 \text{ psi})]^{1/2} + 2 \text{ inches}$$

$$= 12.8 \text{ inches} < 18 \text{ inches} \quad (\text{Equations 17a and 18, this Practice})$$

Wind loads (rather than earthquake) govern footing design. Therefore, no top reinforcement is required.