May 2005

Process Industry Practices
Structural

PIP STE03020
Guidelines for Tank Foundation Designs

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Guidelines for Tank Foundation Designs

Table of Contents

1. Introduction

1.1 Purpose

This Practice provides guidance for the design of tank foundations.

1.2 Scope

This Practice describes the guidelines for design and construction of non-refrigerated, aboveground storage tank foundations. Applicable industry specifications are referenced, and the data required to determine the most appropriate foundation for a tank are presented. In addition, this Practice addresses tank foundations preferred for the different types of soil conditions. This Practice also includes information on settlement and releveling and provides procedures to address these issues.

2. References

Applicable parts of the following Practices, industry codes and standards, and references 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 CVS02010 - *Geotechnical Engineering Investigation Specification*
- PIP STC01015 - *Structural Design Criteria*
- PIP STE05121 - *Anchor Bolt Design Guide*
- PIP STS03001 - *Plain and Reinforced Concrete Specification*
- PIP VECTA001 - *Tank Selection Guide*
- PIP VESTA002 - *Atmospheric Storage Tank Specification (in Accordance with* API Standard 650*)*
- PIP VEDTA003 - *Atmospheric Storage Tank Data Sheet and Instructions (in Accordance with* API Standard 650)

2.2 Industry Codes and Standards

- American Concrete Institute (ACI)
  - ACI 201.2R-01 - *Guide to Durable Concrete*
  - ACI 318 - *Building Code Requirements for Structural Concrete*
  - ACI 350R - *Environmental Engineering Concrete Structures*
- American Petroleum Institute (API)
  - API 650 - *Welded Steel Tanks for Oil Storage* (including Appendices B, E, F, and I)
  - API 653 - *Tank Inspection, Repair, Alteration, and Reconstruction*

- API 2000 - *Venting Atmosphere and Low-Pressure Storage Tanks: Nonrefrigerated and Refrigerated*

- American Society for Testing and Materials (ASTM)
  - ASTM A185 - *Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete*
  - ASTM D1751 - *Standard Specification for Preformed Expansion Joint Filler for Concrete Paving and Structural Construction*

- American Water Works Association (AWWA)
  - AWWA D100 - *Welded Steel Tanks for Water Storage*

2.3 Other References

- Duncan, J. M., and D'Orazio, T. B., *Stability of Steel Oil Storage Tanks, Journal of Geotechnical Engineering*, Vol. 110, No. 9, September 1984
- F. A. Koczwara, *Simple Method Calculates Tank Shell Distortion*, Hydrocarbons Processing, August 1980
- W. Allen Marr, Jose A. Ramos, and T. William Lambe, *Criteria for Settlement of Tanks*, Journal of Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 108, August 1982
- Young, W. C., *Roark's Formulas for Stress and Strain*, sixth edition., McGraw-Hill, January 1989

3. Definitions

contract documents: Any and all documents, including design drawings, that the purchaser has transmitted or otherwise communicated, either by incorporation or by reference, and made part of the legal contract agreement or purchase order agreement between the purchaser and the supplier

owner: The party who owns the facility wherein the tank foundation will be installed

purchaser: The party who awards the contract to the supplier. The purchaser may be the owner or the owner's authorized agent.

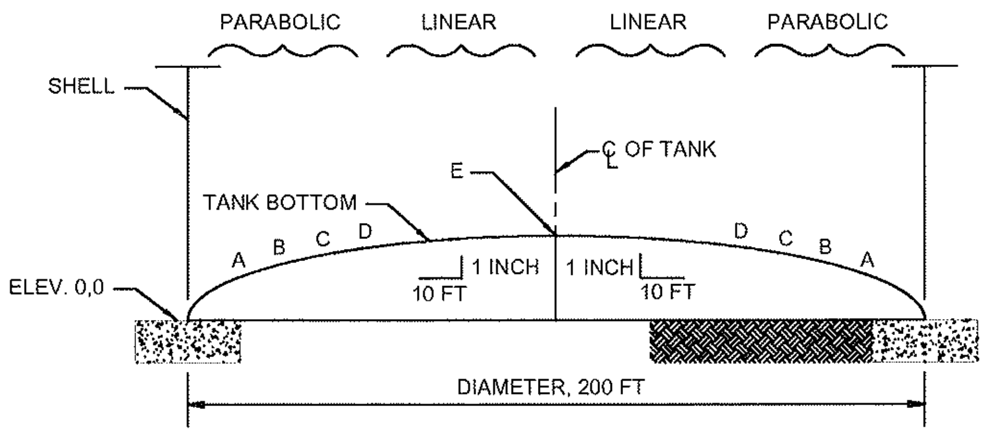
supplier: The party responsible for installing the tank foundation including work executed through the use of sub-contractors

4. General Design Considerations

4.1 Differential Settlement Tank Bottom Designs

4.1.1 Cone-Up Bottom

4.1.1.1 Figure 1 shows a cone-up tank bottom configuration designed to compensate for differential settlement.



TANK BOTTOM COORDINATES		
POINT	DISTANCE FROM TANK SHELL (FT)	HEIGHT ABOVE ELEVATION 0,0 (FT)
A	10	0.71
B	20	1.28
C	30	1.73
D	40	2.00
E	100	2.50

Figure 1. Cone-Up Tank Bottom Configuration

4.1.1.2 As shown in Figure 1, tank bottom plates are placed in a cone-up configuration to compensate for differential settlement. The tank bottom layout shown in Figure 1 is specific for a site with potential differential settlement. The cone-up bottom design can be applied to other sites where large differential settlement is anticipated.

4.1.1.3 The curve shown in Figure 1 is the maximum recommended; steeper slopes can cause the bottom plate to crease. The parabolic portion of the tank bottom layout is defined by considering soil conditions, tank diameter, and tank height. A qualified geotechnical consultant should assist in determining the proper design parameters for such projects. Other configurations can be used.

4.1.1.4 Additional geotechnical guidance can be found in *PIP CVS02010*. See *PIP VECTA001* for guidance on selection of test boring locations.

4.1.2 Cone-Down Bottom - Center Sump Design

4.1.2.1 Another design solution for tank settlement is to design the tank bottom with a minimum downward slope of 1 inch in 10 ft and a center sump and siphon water draw.

4.1.2.2 Although the permissible differential settlement for the cone-down bottom configuration is less than that for a cone-up bottom, the disadvantages related to draining the cone-up bottom are avoided. A cone-down bottom assures good drainage to the center sump even if the tank settles.

4.2 Load Types and Applications

Tank foundations should be designed for the following loads and forces where they exist.

4.2.1 Dead Load

Dead load is the weight of the metal (shell, roof, bottom plates, access ladders, platforms, nozzles, manways, roof support columns, etc.)

4.2.2 Product Load

Product load is the weight of the stored product. Maximum design liquid level and specific gravity should be used to calculate this weight. Test liquid level and test fluid specific gravity should be considered if different from normal stored product.

4.2.3 Vapor Space Design Pressure and Pneumatic Test Pressure

4.2.3.1 Internal pressure on the roof and surface area of the contents is identical; however, the bottom plate (typically 1/4 inch thick, lap fillet welded) is not structurally capable of transferring the vapor pressure to the shell to counterbalance the upward pressure from the roof.

4.2.3.2 Foundations for tanks subjected to internal pressures should be designed to resist the uplift forces in accordance with *API 650*, Appendix F.

4.2.3.3 Tanks with internal pressure generally require foundations with anchor bolts. If anchor bolts are required, see *PIP VESTA002* and *PIP VEDTA003*, item 13, for minimum number and size of anchor bolts. See *PIP STE05121* and contract documents for additional requirements.

4.2.4 Snow Load

For tanks in snow regions, the weight of snow should be considered in the foundation design. Snow load should be calculated in accordance with *PIP STC01015.*

4.2.5 High Temperature

Tanks that store hot products are subjected to temperature variations that can lead to deformations or movements. In the tank foundation, details should be incorporated that allow the tank to move and protect the foundation concrete.

4.2.6 Wind Load

Tank foundations should be designed to resist wind loads. This is particularly important for tanks that may sit empty or only partly filled. Wind loads should be calculated in accordance with *PIP STC01015*.

4.2.7 Earthquake Load

Earthquake-induced lateral forces can cause a tank to tip, overturn, or slide.

4.2.7.1 Earthquake forces should be calculated in accordance with *PIP STC01015* and *API 650*, Appendix E.

4.2.7.2 If the tank does tip on edge, the flexible tank bottom diagonally opposite can lift only a small amount of contents to resist the seismic overturning force. The force of tipping subjects the foundation area under the shell to large vertical compressive forces.

4.2.7.3 The weight of the tank plus its contents and the tank's height-to-diameter (H/D) ratio affect the tank's ability to resist overturning.

a. Small-diameter tanks are more susceptible to overturning than are large-diameter tanks because the small-diameter tanks typically have greater H/D ratios.

b. To verify tank stability, the foundation weight should be added to the tank's shell weight, Wt (see *API 650*, Appendix E), and the tank should be analyzed as unanchored.

c. Unless determined otherwise, the tank should be assumed to be flexible and the foundation should be designed for the full uplift forces.

d. Adjusting the H/D ratio is the preferred method to prevent overturning. Tanks can also be anchored, but this method is not recommended in larger tanks.

4.2.7.4 In seismically active areas, the soil stability should be investigated. The tank site should be analyzed to determine the potential for liquefaction or sliding during an earthquake. This information should be included in the soils investigation report.

4.2.8 Shear Loads

4.2.8.1 Tank stability should be investigated by the geotechnical engineer as a primary issue in tank foundation performance.

4.2.8.2 Punching shear is evaluated when determining the width of the foundation.

4.2.8.3 Edge shear and base shear factors of safety are computed by the methods shown in *Stability of Steel Oil Storage Tanks*.

4.2.8.4 The factors of safety for punching shear, edge shear, and base shear should be greater than 1.5.These safety factors assume that soil conditions under the foundation become evident with a boring/cone penetrometer sounding to a depth of one-fifth the tank diameter every 30 ft of circumference.

4.2.8.5 If the boring spacing is greater than 90 ft, the factors of safety should be 2.0 or greater.

4.2.8.6 Some standards have more stringent safety factors for edge, base, and punching shear.

4.2.8.7 At least one sounding, preferably in the center, should be carried to one full tank diameter in soft soils unless the geotechnical engineer determines otherwise.

4.2.8.8 If stiff soils are near the surface, it is advisable to found the base of the foundation in the stiff soils.

4.2.8.9 It is not normally practical to exceed 4 ft to the base of foundation because of limitations for excavation safety.

4.2.8.10 In some instances, soft soils have been over-excavated and replaced with controlled, low-strength material (CLSM) fill with or lean concrete to put the foundation base at 4 ft for forming. Forming is commonly the most expensive item of a ringwall project.

4.2.9 Settlement

4.2.9.1 Tank settlements should be investigated by the geotechnical engineer as another primary issue in tank foundation performance.

4.2.9.2 Total settlement, differential settlement, interior settlement, and edge settlement should be evaluated and reported.

4.2.9.3 The owner and owner's engineer should provide the geotechnical engineer with information on tank dimensions and expected product and hydrostatic test loading conditions.

4.2.9.4 In some soil conditions, several iterations of settlement evaluations may be needed to arrive at a satisfactory and cost-effective design.

4.3 Foundation Type Selection

Foundation types should be selected on the basis of tank size, site conditions, and environmental requirements.

4.3.1 Tank Size

4.3.1.1 For large tanks (50-ft diameter or greater), concrete ringwall (preferred) or crushed stone ringwall should be used.

4.3.1.2 For mall tanks (20-ft in diameter or less), concrete slab foundation (preferred) or compacted granular fill foundation should be used.

4.3.1.3 For medium tanks (20- to 50-ft diameter), the type of foundation should be at the discretion of the foundation design engineer.

4.3.2 Site Conditions

4.3.2.1 Selecting the appropriate tank foundation depends greatly on the type of soil under the specific tank site. In some instances, large fixed roof tanks can be supported directly on properly prepared high-quality native material. This method should be chosen only if recommended by the soils consultant. Pile-supported concrete slab foundations are used for tanks on poor soils, regardless of the tank size.

4.3.2.2 The dimensions of tanks in high-risk earthquake or wind zones should be proportioned to resist overturning forces, or the tanks should be anchored. In frost regions, extend tank foundations 1 ft below the frost line to prevent frost heave.

4.3.3 Environmental Requirements

4.3.3.1 Environmental requirements are determined by local environmental standards and requirements. Consult with local environmental specialists for recommendations and requirements.

4.3.3.2 Secondary containment, leak detection systems, and cathodic protection should be installed if possible on tanks handling chemicals that could contaminate groundwater if spilled. These systems can also be installed on existing tanks during bottom replacement.

4.3.4 Tank Foundation Summary

Table 1 summarizes foundation types, lists the advantages and disadvantages of each type, and makes specific recommendations.

Table 1. Tank Foundation Summary

Foundation Type	Advantages	Disadvantages	Recommendations
Concrete Ringwall Circular wall is centered continuously under shell circumference.	1. Provides level surface for shell construction 2. Minimizes edge settlement 3. Easy leveling for tank grade 4. Minimizes moisture under tank 5. Retains fill under tank and prevents loss due to erosion 6. Distributes concentrated shell load well 7. Can use cathodic protection 8. Provides greatest assurance of meeting elevation tolerances around tank circumference 9. Better able to transfer shell loads to the supporting soil 10. Minimizes edge settlements and consequently shell distortions—very important problems to avoid for trouble-free operation of tanks with floating roofs	1. Can be expensive, depending on location 2. May not be suitable for tanks on poor soils. Check with foundation specialist. 3. Ringwall must be reinforced. 4. Anchoring of tanks against earthquake overturning is not practical and requires special design.	Preferred foundation type for tanks larger than 20 ft in diameter. Can also be used for small-diameter tanks if anchorage is not required. Use on good soils or properly prepared intermediate soils. Concrete ringwall is the preferred foundation for the following: a. All large tanks b. Tanks where the surface soil is noncohesive, such as loose sand c. Tanks where significant settlement is anticipated d. All floating roof tanks larger than 30 ft in diameter to protect against differential settlement-caused problems with annular space and tank seal
Crushed Stone Ringwall	1. Less expensive than concrete ringwall 2. Good concentrated shell load distribution to weaker soils below 3. Construction material typically readily available 4. Can make use of cathodic protection	1. Tank cannot be anchored against earthquake overturning. 2. Greater care is required for preparation of tank grade. 3. Foundation material is subject to washout. 4. Not suitable for poor soils 5. May cause increased under-tank pitting at points where tank bottom contacts stones. Water and corrosive salts can collect between the stones and cause increased pitting rates. A concrete ringwall will generally cause less bottom-side corrosion where it contacts the tank bottom.	Use if concrete for ringwall is not readily available or if cost of construction is high. Use on good soils or properly prepared intermediate soils. This type of foundation, though not as desirable as a concrete ringwall foundation, is an acceptable alternative, especially in areas with high-quality soil and if concrete is either not readily available or is costly.

(Table 1 continues on next page.)

(Table 1, continued)

Foundation Type	Advantages	Disadvantages	Recommendations
Concrete Slab	1. Provides all the advantages of the ringwall 2. Provides level surface for shell and bottom construction 3. Minimizes differential settlements 4. Good concentrated shell and uniform load distribution 5. Does not require separate bottom support pad 6. Can be designed to allow for tank anchorage against wind and earthquake overturning 7. Can easily incorporate leak detection and containment 8. Low corrosion rate	1. Relatively expensive, especially for large tanks 2. Shifting and settling on poor soils may cause slab to crack. 3. Cannot use cathodic protection	Use for small tanks if leak detection and containment are required. Not recommended for tanks larger than 20-ft diameter because of cost Use on good soils or properly prepared intermediate soils.
Compacted Granular Fill	1. Relatively inexpensive 2. Easy to construct 3. Construction material readily available	1. Limited to small tanks on good soils 2. Tank cannot be anchored against wind and earthquake overturning. 3. Foundation material is subject to washout.	Use on good soils only.
Pile Foundations	1. Minimizes total and differential settlement 2. No separate bottom pad required 3. Allows for tank anchorage against wind and earthquake overturning 4. Leak detection and containment can be incorporated.	1. Most expensive foundation type 2. More complex design than other types 3. Good soils information essential 4. Cathodic protection more difficult to install	Use for all tank foundations on poor soils where no other foundation type is feasible.

5. Foundation Types Design Configurations

5.1 Concrete Ringwall

5.1.1 Common design practice has been to proportion the concrete ringwall so that the soil pressure under the ringwall equals the soil pressure under the confined earth at the same depth as the bottom of the ringwall. This common practice of balancing soil pressures underneath ringwall and foundation pad at the same depth is an attempt to prevent punching shear. The distribution of the soil reaction under the ringwall is trapezoidal and changes with product load, and thus precise balancing is impossible. Tank stability issues including punching shear, edge shear, and base shear control the design of the ringwall section. Settlement issues control the diameter and height of the tank and thus the design of the foundation. Therefore, the ringwall foundation should be designed using the recommendation of the geotechnical engineer to provide adequate factors of safety for stability and allowable settlement of the tank.

5.1.2 Ringwalls should be 12-inch minimum wide with 3-inch minimum above the lowest adjacent grade if paved and 6-inch minimum if unpaved, after predicted settlement.

5.1.3 If leak detection pipes are used, they should be 3 to 6 inches above grade, which will put the top of concrete ringwall about 12 inches above grade. Alternately, leak detection pipes could also be below grade and drain to a pit.

5.1.4 The bottom of a ringwall should be 6 inches minimum below the frost line and 24 inches minimum below grade unless required otherwise by the geotechnical investigation. A greater depth may be required for loose sand.

5.1.5 The minimum concrete strength should be 3,000 psi at 28 days.

5.1.6 Concrete and reinforcement should be specified in accordance with *ACI 318* and *API 650*, Appendices B, E, F, and I.

5.1.7 Concrete ringwalls should be reinforced to reduce shrinkage cracks and to resist hoop tension, which is caused by lateral earth pressure inside a ringwall from the product surcharge and applicable tank dead load, such as from the tank bottom plate and roof columns.

5.1.8 The lateral earth pressure should be assumed to be 50% minimum of the vertical pressure from fluid and soil height, unless determined otherwise by proper geotechnical analysis. If a granular backfill is used, a lateral earth pressure coefficient of 30% may be used.

5.1.9 Passive pressure on the outside of the ringwall should not be included in the calculations.

5.1.10 Except for hot tanks, a 1/2-inch-thick minimum, asphalt-impregnated board should be placed, in accordance with *ASTM D1751*, on top of the wall directly underneath the shell annular plate.

5.1.11 The space within the ringwall should be backfilled with compacted granular fill capable of supporting the tank dead load and the product surcharge load. Backfill should be select material of such size and gradation as to be easily compacted and have good drainage characteristics. Generally, material meeting the requirements for roadway base in local areas is acceptable backfill.

5.2 Crushed Stone Ringwall

5.2.1 Crushed stone ringwalls should consist of crushed gravel or crushed stone 1/2 to 1 inch in diameter.

5.2.2 A crushed stone ringwall base should be wide enough to distribute the shell loads to the underlying soil without exceeding the allowable bearing capacity.

5.2.3 The ringwall base width and depth below the bottom of the tank annular plate should be determined in accordance with the recommendations of the geotechnical consultant.

5.2.4 The depth on a crushed stone ringwall should be 2 ft minimum.

5.2.5 All other ringwall dimensions should be in accordance with *API 650*, Appendix B, except that the berm outside the tank should be in accordance with Section 6.5 of this Practice.

5.2.6 The space within the crushed rock ringwall should be backfilled with compacted granular fill in accordance with Section 5.1.11.

5.3 Concrete Slab Foundation

5.3.1 Concrete slab tank foundations can be used to support small unanchored or anchored tanks. The concrete slab can provide an outstanding level, uniform tank support surface and allows tank anchoring with conventional anchor bolts. For small production tanks, precast concrete slabs transported to site by truck may offer a quick, simple, and cheap foundation.

5.3.2 The slab should be thick enough to develop the anchor bolt forces and rigid enough to transfer the tank loads to the soil without cracking.

5.3.3 Structural concrete should be designed in accordance with *PIP STC01015* and *PIP STS03001*.

5.3.4 The concrete slab should be reinforced to reduce shrinkage and to resist shear and bending moments produced by soil-bearing pressures.

5.3.5 Reinforcement can consist of deformed steel bars or deformed welded wire fabric.

5.3.6 The concrete slab should be heavy enough to resist overturning forces with a safety factor in accordance with *PIP STC01015*.

5.4 Compacted Granular Fill Foundation

5.4.1 Unanchored small tanks can be supported on compacted granular fill placed directly over native material.

5.4.2 The granular fill should be 1-ft minimum deep.

5.4.3 Protection against erosion can be accomplished in one of two ways:

a. A 3-ft-wide shoulder and berm built around the tank

b. A steel band placed around the periphery of the tank

5.4.4 The steel band method also confines the fill and prevents sloughing of loose, non-cohesive surface soil.

5.4.5 A construction detail for a compacted granular fill foundation with a steel band is shown in Figure 2.

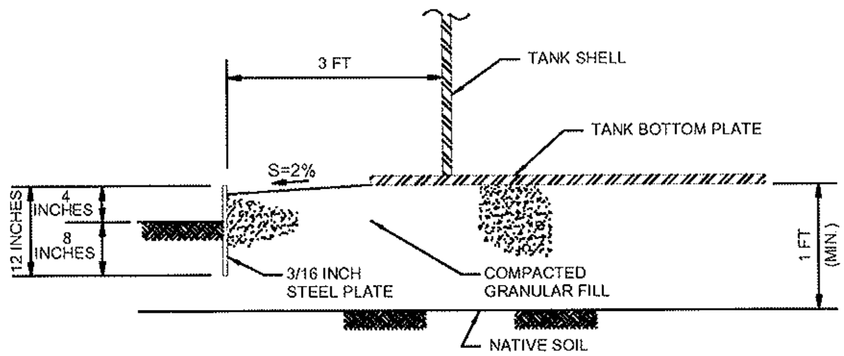


Figure 2. Granular Fill Foundation with Steel Plate Band

5.4.6 If the native soil does not drain, the fill could stay full of water and cause increased corrosion; therefore, the native soil should be sloped for drainage; or cathodic protection should be used to protect the bottom.

5.5 Pile-Supported Concrete Foundation

5.5.1 If tank loads and soil conditions do not economically permit use of any of the previously discussed foundation types, a pile-supported foundation may be the only practical alternative.

5.5.2 The following procedure is provided for designing pile-supported foundations:

a. Make a soils investigation to determine groundwater levels, allowable pile loads, and required pile lengths.

b. Calculate the loads and estimate the total number of piles.

c. Determine type, capacity, and length of piles.

d. Establish pile spacing and pile group effect.

e. Design the pile cap and concrete slab.

f. Check pile uplift and lateral loads resulting from wind or earthquake.

5.6 Ringwall Foundation Design Procedures

5.6.1 General

5.6.1.1 Because of the large compressive forces in the tank shell, the ringwall design is critical. This section describes design procedures developed in accordance with *API 650* and *ACI 318* that should be used to design a ringwall foundation for a tank.

5.6.1.3 Appendix A provides design examples for a tank located in a high-seismic area and for a tank located in a low-seismic area.

5.6.1.2 See Figure 3 for the loading and assumptions that should be used to design a ringwall foundation.

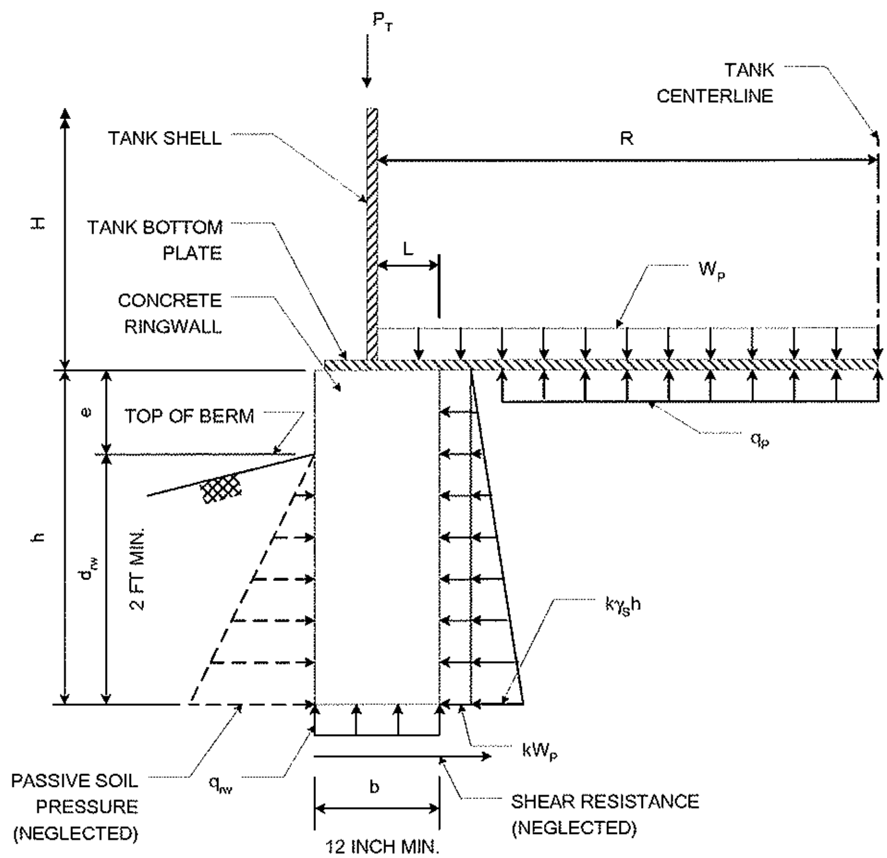


Figure 3. Ringwall Loading for Unanchored Tank

Legend:

R = tank radius, ft

H = tank height, ft

b = width of ringwall, ft (should be 12 inches minimum)

h = height of ringwall, ft

drw = depth of ringwall below grade, ft

L = distance from tank shell to inside edge of ringwall, ft

γc = unit weight of concrete, pcf

γs = unit weight of soil, pcf

e = distance of top of ringwall from top of berm, ft

k = coefficient of lateral earth pressure in accordance with Section 5.1.8

PT = total load on tank shell, lb/ft

Wp = product load on tank bottom, psf

q_p = net soil bearing under tank inside ringwall, psf

q_{rw} = net soil bearing under ringwall, psf

q^a = net allowable soil bearing under ringwall determined by the geotechnical engineer, psf

M_T = applied uniform twist moment on the ringwall, kip-ft/ft

T_h = hoop tension, kips

fy = yield strength of reinforcing steel, psi

f'c = compressive strength of concrete, psi

5.6.2 Foundation Sizing

5.6.2.1 The width of the ringwall foundation should be determined on the basis of the allowable soil-bearing pressure.

5.6.2.2 For load combinations including hydrotest, wind, or seismic loads, the allowable soil-bearing pressure may be increased in accordance with the recommendations of the geotechnical engineer.

5.6.2.3 If soil bearing controls the design, the required width of the ringwall foundation may be determined using the following equation:

$$b = \frac{P_T + W_P(L)}{q^a + (h-e)\gamma_S - h\gamma_C}$$

5.6.2.4 If soil bearing controls the design, the value of L should be minimized to obtain an economical ringwall design. This design recommendation often results in most of the concrete ringwall width being located outside the tank shell. The minimum inside edge distance (L) should be in accordance with Table 2:

Table 2. Minimum Inside Edge Distance

Tank Diameter D (ft)	Minimum Inside Edge Distance L (inches)
$D \leq 80$	4
$80 < D < 150$	5
$D \geq 150$	6

5.6.2.5 The minimum values of L shown in Table 2are based on experience and should provide sufficient edge distance to

a. Compensate for concrete construction tolerances.

b. Prevent spalling of the concrete from high bearing pressures at the tank wall.

5.6.2.6 If uplift controls the design (anchored tank), sufficient counterbalancing weight (the weight of the foundation, the weight of soil over the foundation, and in the case of seismic loading, weight of tank product over the foundation) should be provided to completely resist the uplift. This design recommendation typically results in most of the concrete ringwall width being located inside of the tank shell. Anchor bolts should be placed with enough edge distance to the outside of the ringwall to develop the strength of the bolt. The anchor bolts should also be placed inside of the outer face of hoop steel to facilitate construction.

5.6.2.7 If the ringwall foundation width is greater than its depth, the design should consider the foundation's behavior as an annular slab with flexure in the radial direction.

5.6.3 Hoop Tension

5.6.3.1 Axial tension is generated in the ringwall foundation by lateral earth pressure. The lateral earth pressure is to the result of the product surcharge and the backfill within the ringwall foundation.

5.6.3.2 The counterbalancing effect of passive pressure on the outside of the ringwall foundation usually should be ignored because of the difficulty in assuring its reliability.

5.6.3.3 The unfactored hoop tension force is determined using the following equation:

$$T_h = R\,h\,k\left(W_P + \frac{\gamma_S\,h}{2}\right)$$

5.6.3.4 The required hoop steel should be determined using the factored hoop tension force and in accordance with *ACI 318* using the following equation:

$$A_S = \frac{1.6T_h}{0.9f_y}$$

5.6.4 Twist

5.6.4.1 Eccentric loadings from the tank shell, product over the ringwall, anchor bolts, and soil reaction beneath the ringwall foundation can act to create a twisting moment (MT) on the ringwall foundation.

5.6.5.2 For the cases of dead load plus fluid load and dead load plus internal pressure, a twist moment should be applied uniformly around the ringwall foundation, as shown in Figure 4.

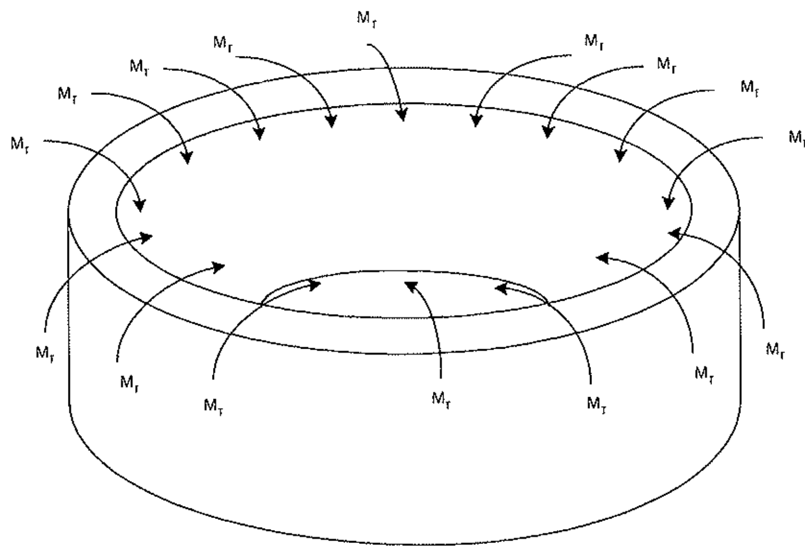


Figure 4. Uniform Twist Moment

5.6.4.3 For the loading case in Figure 4, the twist moment tend to push one end of the ringwall outward, thus inducing tension, and tend to push the other end of the ringwall inward, thus inducing compression. Additional hoop reinforcing steel should be added to the tension region.

5.6.4.4 The twist moment should be calculated as shown in Figure 5. The dimension, x_bar, is the distance between the centerline of the ringwall and the center of the applied load as shown in Figure 5.

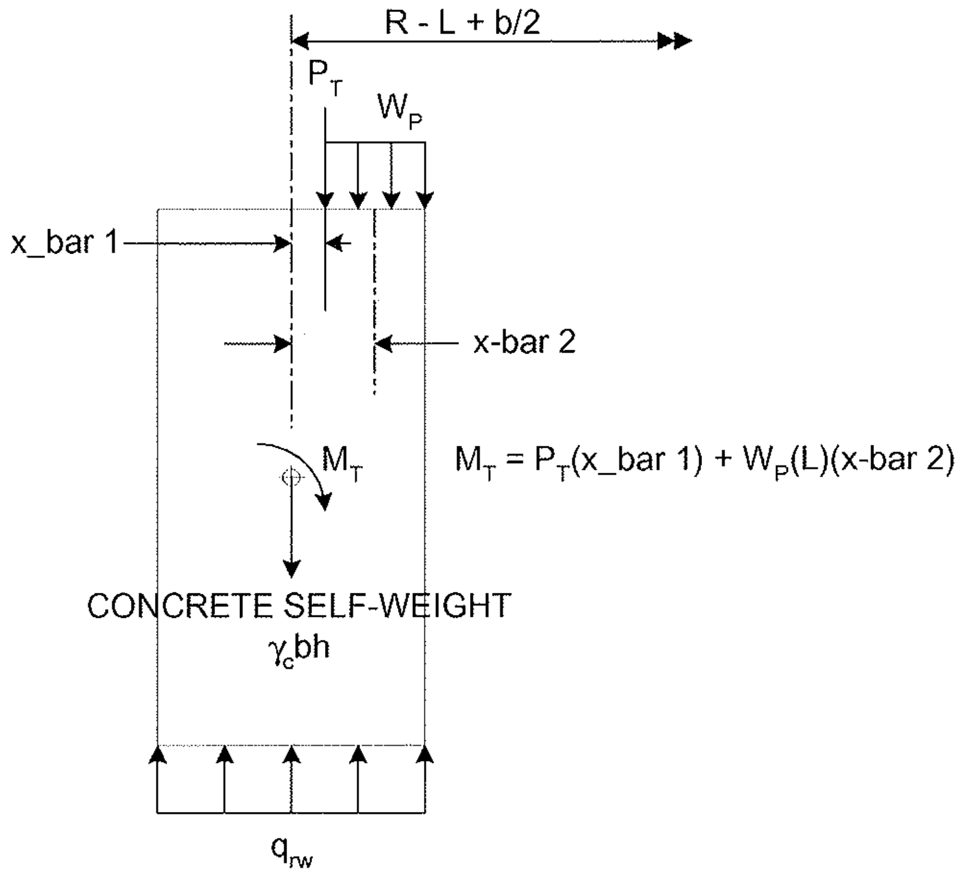


Figure 5. Calculation of Twist Moment

5.6.4.5 Typically, unanchored tank ringwall foundations require additional twist hoop steel in the bottom of the ringwall, and anchored tank ringwall foundations require additional twist hoop steel in both the top and bottom of the ringwall.

4.6.5.6 The ringwall can be designed for an equivalent bending moment about the horizontal axis of the ringwall in accordance with the following equation taken from Section 10.9 of *Roark's Formulas for Stress and Strain*:

$$M = M_T\left(R - L + \frac{b}{2}\right)$$

5.6.4.7 For load cases including seismic or wind loads, the load distribution on the ringwall becomes significantly more complex, as shown in Figure 6.

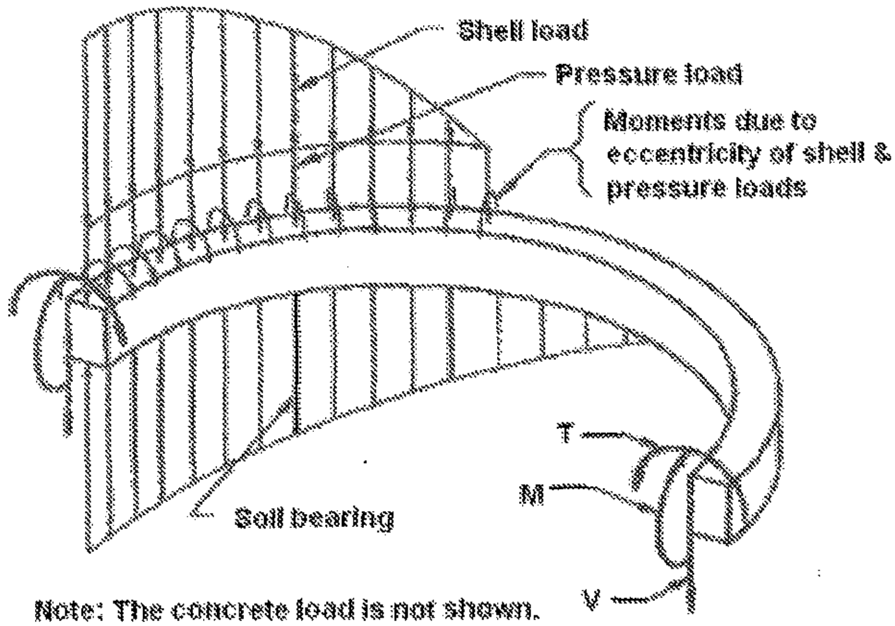


Figure 6. Load Distribution under Seismic or Wind

5.6.4.8 For simplicity of calculation, the effect of the twist moment for the more complicated seismic and wind load distributions should handled in the same manner as that for the uniform distribution case. The peak value of the twist moment is used in this case.

5.6.4.9 The ringwall foundation design examples printed in Appendix A are based on a 1-ft rectangular section of the ringwall foundation. A more refined analysis can be made using a 1-ft wedge-shaped section of the ringwall foundation, or a more elaborate analysis can be made using a curved beam analysis. These refined analyses are beyond the scope of this Practice.

5.6.4.10 A key point to understand about the effect of twist on a ringwall foundation is that the twist moment does not induce torsional stresses into the ringwall. In Figure 6, the values of T (torsion) and V (shear) at the boundaries of the curved beam are equal to zero. The end conditions of the curved beam shown in Figure 6 are approximated by using supported and slope-guided conditions at both ends of the curved beam.

5.6.5 Base Shear and Sliding

5.6.5.1 Traditionally, ringwall foundations are not designed for the additional overturning moment from the base shear (wind or seismic) applied at the top of the ringwall. Most of the additional overturning moment does not develop because of the passive pressure resistance provided by the soil outside of the ringwall foundation and friction between the soil and the vertical sides of the ringwall. What little additional overturning moment that does develop is not significant and is ignored in the design.

5.6.5.2 If the ringwall foundation has a significant projection above grade (i.e., average height above grade greater than 2 ft), the additional overturning moment from this projection should be considered in the design.

5.6.5.3 Except under extremely high seismic forces (horizontal and vertical) and/or unusual soil conditions, ground-supported flat bottom tanks and their foundations do not slide. If sliding of the foundation should be checked, friction between the foundation (including soil within the foundation) and the soil combined with the passive pressure resistance of the soil outside the foundation may be used to resist sliding.

5.6.6 Minimum Reinforcing Steel Requirements

5.6.6.1 In accordance with *API 650* Appendix B and *ACI 318* Chapter 14, the following minimum reinforcing steel ratios should be provided to resist temperature effects and shrinkage:

a. Minimum ratio of vertical reinforcement area to gross concrete area:

(1) 0.0012 for deformed bars not larger than No. 5 with a specified yield strength not less than 60,000 psi; or

(2) 0.0015 for other deformed bars

b. Minimum ratio of horizontal reinforcement area to gross concrete area:

(1) 0.0020 for deformed bars not larger than No. 5 with a specified yield strength not less than 60,000 psi; or

(2) 0.0025 for other deformed bars

5.5.6.2 Vertical and horizontal reinforcement should not be spaced farther apart than 18 inches.

5.5.6.3 In accordance with *ACI 350R-89* Section 2.5, concrete sections 24 inches or thicker may have the minimum temperature and shrinkage reinforcement at each face based on a 12-inch thickness.

5.6.7 Special Ringwall Foundation Considerations for Seismic Loads

5.6.7.1 Unanchored (also called self-anchored) tanks produce significantly higher toe pressures to satisfy equilibrium than do mechanically anchored tanks under seismic loads. Using the calculated maximum toe pressure in the tank shell to satisfy equilibrium on unanchored tanks produces impractical ringwall dimensions. Some yielding of soil (settlement) may occur under the shell, requiring re-leveling of the tank after a seismic event. The foundations under tanks, even tanks resting directly on earth foundations, have fared well under seismic loadings. Therefore, the seismic loading does not alter the ringwall foundation design criteria or provide justification for increased foundations. This is not true for slab and pile cap foundations, which should be designed for the maximum toe pressure.

5.6.7.2 Tank ringwall foundations are normally designed for the ringwall moment in accordance with *API 650* or *AWWA D100*. The ringwall moment is the portion of the total overturning moment that acts at the base of the tank shell perimeter. The total overturning moment, also known as the slab moment, is used to design slab and pile cap foundations. While the difference between the ringwall moment and the slab moment can be resolved into an equivalent liquid pressure acting on the bottom of the tank, this additional pressure should not be used to design ringwall foundations.

5.6.7.3 Ringwall foundations for tanks should consider forces from vertical seismic accelerations if these forces are specified in addition to forces from horizontal seismic accelerations. The maximum vertical seismic force does not occur simultaneously with the maximum horizontal seismic force. For combining horizontal and vertical seismic forces, *API 650* combines 100% of the seismic load from horizontal acceleration with 40% of the seismic load from vertical acceleration. If vertical seismic accelerations are applicable, the product load directly over the ringwall should be applied as follows:

a. If used to resist the maximum anchor uplift on the foundation, the product pressure should be multiplied by a factor of ($1-0.4A_v$, where A_v is the maximum vertical seismic acceleration adjusted for use with allowable stress design methods). The dead load should also be reduced by this same factor. The ringwall foundation should be designed to resist the eccentric loads with or without the vertical seismic acceleration.

b. If used to evaluate bearing, the product pressure over the ringwall should be multiplied by a factor of ($1 + 0.4A_v$). The dead load should also be increased by this same factor. The ringwall foundation should be designed to resist the eccentric loads with or without the vertical seismic acceleration.

6. Special Design Considerations

6.1 *API 650* Tolerances

6.1.1 To achieve the *API 650* tolerances for tank erection, the following note should be shown on the ringwall foundation drawing:

"The top of the concrete ringwall should be level within ±1/8 inch in any 30 ft of the circumference and within ±1/4 inch in the total circumference measured from the average elevation."

6.1.2 If a concrete ringwall is not provided, the foundation under the shell should be level within ±1/8 inch in any 10 ft of the circumference and within ±1/2 inch in the total circumference measured from the average elevation.

6.2 Bottom Support Pad

6.2.1 Depending on the choice of corrosion protection and leak detection method, the area within the ringwall and above the aggregate backfill may be covered with the following:

a. Reinforced concrete slab

b. Sand pad

6.2.2 To prevent corrosion, the shoulder should be lowered around the tank and water should be properly drained away from the tank.

6.2.3 Table 3 provides a summary of the bottom pad types and specific recommendations regarding leak detection and containment and corrosion protection.

Table 3. Bottom Pad Types

Bottom Support Pad Type	Incorporation of Leak Detection and Containment	Incorporation of External Cathodic Protection	Comments
Reinforced Concrete Slab	Can be incorporated Can easily accommodate leak detection	Not required Will not permit cathodic protection	Reinforcement is required. Use if leak detection and containment are required. Do not use if cathodic protection is required. Outstanding support surface for the bottom plate. Do not use if the anticipated differential settlement is more than 1 inch in 10 ft.
Plain Sand Pad	Can be incorporated	Easiest to incorporate	The sand can shift causing voids and low spots. Easy to construct; difficult to maintain while installing bottom. Laying of the bottom can disrupt the contour of the sand. Although shifting sand is a concern, the problems caused by shifting sand are generally less than those caused by a concrete pad on shifting ground, because cracking and break-up of the concrete are serious problem. Any oil added to the sand can represent pollution and potential groundwater contamination. Use if leak detection and containment and/or cathodic protection are required.
Granular Fill	Leak detection very difficult or impractical.	Can be incorporated	See "Tank Foundation Summary," Table 1.

6.2.4 The pad for reinforced concrete slabs should

a. Be 5 inches minimum thick

b. Rest on 4 inches minimum thickness of sand or compacted fill cushion

6.2.5 The pad for plain sand or granular fill should 4 inches minimum thickness of clean, salt-free sand.

6.2.5.1 Sand causes much less corrosion than either gravel or crushed stone.

6.2.5.2 Some localities allow the use of oil in sand as a corrosion inhibitor. Oil provides only little corrosion resistance, and in some cases actually increases corrosion rates. Any oil added to the sand can represent pollution and potential groundwater contamination.

6.2.5.3 Rather than using oil as a corrosion inhibitor, consider installing cathodic protection in the sand pad.

6.3 Hot Tanks

6.3.1 General

6.3.1.1 Guidelines are provided in this section for the following:

a. Hot tank bottom foundation design

b. Leak detection

c. Leak containment

6.3.1.2 Although the principles are applicable to any hot tank, the designs have been tailored for tanks storing hot asphalt products in the temperature range of 200°F to 600°F.

6.3.1.3 These guidelines do not address tanks exposed to a large temperature gradient or frequent heating and cooling cycles. For such conditions, special consideration should be given to fatigue, thermal expansion, and creep.

6.3.1.4 The guidelines provided in this section have the following goals:

a. Minimize the costs for design, installation, and maintenance

b. Provide a high-quality installation that is safe, reliable, and easy to maintain

c. Provide standardized designs that have the flexibility to 6meet local conditions and requirements

d. Include tank bottom retrofits in the design standards

6.3.2 Under-Tank Temperatures

6.3.2.1 In a temperature distribution study, high temperatures were found to exist several feet below the bottom of a hot tank. Initial temperature profiles will vary from site-to-site because of factors such as presence of moisture or different soil thermal conductivity.

6.3.2.2 After a tank is put into hot service, months or years may pass before the ground temperatures reach steady-state conditions. Eventually, however, high temperatures will extend several feet below the tank's foundation.

6.3.2.3 In actual field tests, temperatures of 160°F at a depth of 30 inches below some tanks after a relatively short period of service.

6.3.2.4. Wooden piles have been found at a hot tank foundation site to be charred to a depth of several feet below the tank's concrete slab. (Wood piles are not recommended for hot tank foundations.)

6.3.2.5 If temperatures of 212°F and above are combined with high ground water levels, vent pipes should be installed in ringwall foundations.

6.3.3 Under-Tank Insulation

6.3.3.1 Under-tank insulation should not be permitted.

6.3.3.2 To counter the effects of high under-tank temperatures, some designers have suggested using under-tank insulation. Temperature distribution studies indicate, however, that insulation does not reduce steady-state temperatures because the thermal gradient across the insulation should be large for the insulation to be effective.

6.3.3.3 Unless the thermal conductivity of the insulation is much lower than that of the soil, the insulation can not work.

6.3.3.4 Although additional insulation may increase the time required to reach a steady-state condition, the eventual effects of high under-tank temperatures can not be eased.

6.3.3.5 Insulation can also generate other problems, such as increased settlement, moisture entrapment, tank bottom corrosion, and maintenance difficulties.

6.3.4 Environmental Considerations

6.3.4.1 Many regulatory agencies require release-prevention barriers and leak detection devices for tanks, including hot tanks. Release-prevention barriers typically consist of under-tank liners.

6.3.4.2 Materials such as asphalt, typically stored in a temperature range of 350°F to 500°F, or molten sulfur stored above its melting point of 239°F, are solid at ambient temperature. Because these materials would solidify if leaked and because both asphalt and sulfur have been used to pave highways, it is unlikely that any environmental harm would occur from under-tank leaks. For these materials, owners should negotiate a leak containment solution on a case-by-case basis.

6.3.4.3 Liners should be used for hot substances that are liquid at ambient temperature or are toxic if leaked.

6.3.5 Leak Detection and Containment

6.3.5.1 General

1. For leak detection, *API 650* requires tank-bottom leakage be redirected to the tank perimeter where the leakage can be observed. An under-tank liner can both redirect the flow for leak detection and also act as a release-prevention barrier or liner.

2. Clay, concrete, and steel liners have been used for hot tanks. The choice of material should be based on economics, maintenance concerns, and local regulations.
3. For ambient-temperature tanks, plastic liners can be provided for leak detection and containment. However, high temperatures can exist several feet below a hot tank. A double steel bottom (metallic liner) or a concrete liner should be used for temperatures exceeding 250°F.
4. All liners (including plastic liners) should be designed for stock-side temperatures.

6.3.5.2 Clay Liners

1. Clay liners can withstand temperatures greater than 200°F without melting, but they are susceptible to drying and cracking unless kept continuously moist.
2. Clay liners should not be used unless required by law because they degrade if subjected to high under-tank temperatures.
3. High under-tank temperatures drive moisture away causing clay liners to crack.
4. Clay liners should be placed near the water table to keep the clay moist and prevent cracking.
5. A clay liner should be placed inside a ringwall foundation and covered with chloride-free, dry sand before tank construction as shown in Appendix B, Figure B-2.

6.3.5.3 Concrete

1. Concrete may be an under-tank liner or a release-prevention barrier if it meets certain requirements.
2. *ACI 350R-89* lists requirements and recommendations for structural design, materials, and construction of concrete tanks and other reservoirs.
3. Although permeability is not addressed in *ACI 350R-89*, watertightness is addressed. A watertight concrete liner should prevent a release of an environmentally threatening compound; however, local regulators determine what actually constitutes an acceptable release-prevention barrier.
4. To be watertight, the concrete cracking should be controlled by temperature and shrinkage reinforcement in accordance with *ACI 350R-89*.

6.3.5.4 Polymer-Based Liners

1. Polymer-based liners, including high-density polyethylene (HDPE), will melt or stretch and tear apart from the tank's weight or shifting soil. Therefore, plastic liners should not be used for hot tanks unless designed for stock-side temperature.

2. Plastic liners should not be used unless required by law because they degrade if subjected to the high under-tank temperatures.

6.3.5.5 Elastomeric Liners

1. Although most elastomeric liners are reliable only to approximately 250°F, Teflon® can withstand 450°F temperatures.
2. Although the cost is high, heat-seamable PFA Teflon® (available in 60- to 90-mil sheets in 4-ft widths, by 100 or more ft long) can be used.

6.3.6 Concrete at High Temperatures

6.3.6.1 Concrete compressive strength decreases as temperatures increase. Reduction in strength results from temperature, moisture content, loading history, and the type of aggregate used. As the concrete heats up, the aggregate and cement expand at different rates. This, coupled with the different stiffnesses for the aggregate and the cement, creates a complex interaction.

6.4.6.2 For concretes with limestone or gravel aggregate in temperatures up to 600°F, the strength reduction is very small. Concrete with other aggregates, however, may have up to a 40% strength reduction at 600°F.

6.3.6.3 At temperatures greater than 600°F, the cement starts to dehydrate and its strength drops off more dramatically. Therefore, for temperatures higher than 600°F, special types of cement such as alumina cement should be considered. Using alumina cement concrete for tank foundations with tank temperatures less than 600°F is very costly and probably not necessary.

6.4.1.4 For concretes required to tolerate temperatures less than 600°F:

a. Regular concrete with an appropriate strength-reduction factor may be used for foundations.
b. For tank temperatures in the range of 200°F to 400°F, 4,000 psi concrete should be used.
c. For tank temperatures in the range of 400°F to 600°F, 5,000 psi concrete should be used.
d. Although higher strength concrete is used, in both cases, the foundation should be designed using a reduced strength of 3,000 psi to provide an adequate safety factor.

6.3.6.5 Reinforced concrete design should be in accordance with *ACI 318* requirements and ringwall design guidelines as specified in this Practice.

6.3.7 Concrete Mix

6.3.7.1 High-quality concrete should be used with a low water/cement ratio for hot tanks. The following design mixture is recommended:

a. 0.4 water-to-concrete ratio

b. minimum of 490 lb per cubic yard minimum cement

c. 5% minimum entrained air

d. No accelerators (especially accelerators with chlorides)

6.3.7.2 Chloride salts should not be added to the concrete to accelerate hardening. To prevent corrosion, concrete should not exceed 0.15% soluble chlorides in accordance with *ACI 201.2R-01*.

6.3.7.3 Curing procedures should include keeping the new concrete surface damp for the first 7 days minimally.

6.3.7.4 Locally available aggregate should be acceptable because the design should already take into account the reduced concrete strength at high temperatures.

6.3.8 Selecting Foundation Type

6.3.8.1 Appendix B, Figure B-1 provides a chart for selecting a hot tank foundation, taking into consideration the liner, leak detection, and other variables.

6.3.8.2 Single Bottom Designs

1. Single bottom designs with concrete liners and slabs under the tank are shown in Appendix B, Figures B-3 and B-5.

2. Single bottom concrete slabs and/or ringwall foundations are recommended for hot tanks because the slab offers the following advantages:

   a. Provides a release-prevention barrier or liner under the tank.

   b. Reduces the possibility of moisture collecting under the tank bottom. Moisture can accelerate corrosion or cause temperature variations that create high local stresses on the shell-to-bottom welds and the bottom plates.

   c. Provides the opportunity to install leak detection grooves in accordance with *API 650*.

3. The concrete slab should be installed to cover the entire bottom of the tank. The concrete foundation acts as a liner, creating a barrier, which prevents groundwater contamination.

4. The foundation also should include leak detection grooves, which guide the leaking product toward the tank's periphery for easy detection. See Appendix B, Figure B-4.

5. The concrete should be reinforced so that cracks cannot propagate and undermine the concrete's integrity. As with any

other design, temperature steel should be included in the ringwall and concrete slab. Because of thermal gradients, however, additional reinforcing steel should be placed in the circumferential (hoop) direction near the outside edge.

6. If the tank is less than 30 ft in diameter, the integral ringwall-slab design shown Appendix B, Figure B-3 should be less costly and more effective to use. Instead of a ringwall, a slab with thickened edges is used. The required reinforcing, leak detection, and thermal considerations are the same as those for larger tank foundations.

7. Appendix B, Figure B5 includes an expansion joint to accommodate the thermal growth of the slab relative to the ringwall. The temperature range for this design is from 200°F to 600°F. In the configuration shown in Figure B-5, a leak would not be contained but would seep into the secondary containment area. Because a leak can be quickly detected, stopped, and cleaned, however, environmental considerations should not be critical in this configuration.

8. Appendix B, Figure B-6 shows an alternative to a slab under the tank. This design uses a curb to provide more leak containment but is probably no more effective than other designs and probably more costly. However, the use of this design may be required by local authorities.

6.3.8.3 Tank Bottom Replacement or Retrofitting

1. Appendix B, Figure B-7 shows an economical and reliable method for providing a liner and leak detection system for upgrading or replacing the bottom of existing tanks for high-temperature service.

2. A new concrete spacer, 4 to 6 inches minimum thickness, should be poured over the old tank bottom.

3. The concrete liner should be reinforced in accordance with *ACI 350R* to provide watertightness and to prevent excessive cracking.

4. Radial grooves should be added for leak detection.

5. For substances that may not be considered hazardous, such as asphalt and sulfur, welded wire mesh is adequate reinforcement in lieu of rebars because cracking would not create environmental concerns.

6.3.8.4 Double Steel Bottoms Designs

1. A double steel bottom is the preferred method for leak detection/containment.

2. Appendix B, Figure B-8 can be used as guidance for new tanks or for replacing a tank's bottom plate. This design provides

containment in the form of a double steel bottom; the tank bottom closest to the ground forms the liner or release-prevention barrier. The system is built on compacted fill soil.

6.3.9 Anchoring

6.3.9.1 Tanks should be designed with low height-to-diameter (H/D) ratios so that anchoring is not required to resist for the seismic loadings in accordance with *API 650*, Appendix E.

6.3.9.2 If it is not possible to keep tank's H/D ratio low enough (approximately 0.4 to 0.5 in seismic Zone 4), anchors can be required.

6.3.9.3 The anchorage should be designed to accommodate the differential thermal expansion in the radial direction between the tank and the slab.

6.3.9.4 Appendix B, Figure B-9 should be used if a hot tank requires seismic anchorage. This detail allows for the different radial expansions that can occur between the tank and its foundation without generating significant bending stresses in the anchor bolts.

6.3.10 Sumps

6.3.10.1 Emptying a hot tank for cleaning, inspection, maintenance, and repair can be difficult if the contents solidify or become hard to handle at ambient temperatures. Therefore, bottom sumps may be required.

6.3.10.2 In hot tanks the indiscriminate use and design of tank-bottom sumps or appurtenances have led to failures because of thermal expansion of the tank bottom. For sumps or appurtenances to perform reliably and without risk of failure, they should be designed on a case-by-case basis

6.3.10.3 Appendix B, Figure B-10 shows one type of sump design.

6.3.11 Corrosion

6.3.11.1 Corrosion in hot tanks can occur anywhere water contacts a tank bottom plate. Usually, the high under-tank temperatures drive away existing moisture, especially near the tank's center. However, in, locations having frequent rains, a high water table, or frequent flooding, water can remain in contact with the tank bottom.

6.3.11.2 Generally, corrosion is limited to a tank's periphery because that is the only area where water can have lasting contact with the tank shell and bottom.

6.3.11.3 A tank's edge may never become completely dry because of a phenomenon known as moisture pumping. As the water under the tank is heated, it rises, pushing the water above the tank out of the way and drawing more water in to take its place. Moisture pumping can be minimized by placing a tank well above the water table. In addition, a concrete pad or ringwall foundation can create an effective barrier, minimizing moisture pumping.

6.3.11.4 For hot tanks, water in contact with the bottom plate usually turns to steam. Although steam is less corrosive than liquid water, its corrosive effects should be considered.

6.3.11.5 In existing tanks if the chime (the external part of the annular ring) sits in water, severe corrosion can be expected. With the combination of thermal stresses and corrosion, a potential for failure exists at this critical shell-to-bottom joint. The tank perimeter should be excavated and drained to assure that water does not collect around the base.

6.3.11.6 The best way to reduce under-tank corrosion is to keep the tank's underside dry. Raising the tank 4 to 6 inches above the adjacent grade, including the amount for future foundation settlement, should reduce moisture contact and bottom-side corrosion.

6.3.11.7 Cathodic protection under hot tanks should not be specified because the anode's life is greatly reduced at elevated temperatures.

6.4 Small Tanks

6.4.1 Shop-Welded Tanks

6.4.1.1 The size of shop-welded tanks is limited by what can be transported over public highways or railroads.

6.4.1.2 A concrete pad is the preferred foundation for shop-welded tanks. The pad provides a level surface for placing the tank, allows for anchoring the tank if required, and can be used for leak detection.

6.5.1.3 In good soil locations, unanchored small, shop-welded tanks can be supported on compacted granular-fill foundations. A gravel pad does not provide as level a surface as does a concrete pad, but it is structurally adequate. Gravel or sand pads can be subject to surface irregularities during tank placement. Such pads can also shift, causing voids underneath the bottom.

6.4.2 Tanks with Design Pressures to 2.5 psig

6.4.2.1 *API 650*, Appendix F, provides requirements for the design of tanks subject to small internal pressures up to 2.5 psig.

6.4.2.2 If the internal pressure multiplied by the cross-sectional area of the nominal tank diameter is less than the nominal weight of the metal in the shell, roof, and any framing supported by the shell or roof, and if the tank is anchored, the foundation should be sized to resist to uplift from the greatest of the following load conditions:

a. The uplift produced by the design pressure of the empty tank (minus any specified corrosion allowance) plus the uplift from the design wind velocity on the tank

b. The uplift produced by the design pressure of the tank filled with the design liquid (minus any specified corrosion allowance) plus the uplift from the design earthquake on the tank. The effective weight of the liquid should be limited to the inside projection of the ringwall from the tank shell. If a footing is included in the ringwall design, the effective weight of the soil may be included.

6.4.2.3 If the internal pressure multiplied by the cross-sectional area of the nominal tank diameter exceeds the nominal weight of the metal in the shell, roof, and any framing supported by the shell or roof, the foundation should be sized to resist to uplift from the greatest of the following load conditions in accordance with *API 650* Section F.7.5 and the load conditions outlined in Sections 6.4.2.2a and b of this Practice:

a. The uplift produced by 1.5 times the design pressure of the empty tank (minus any specified corrosion allowance) plus the uplift from the design wind velocity on the tank

b. The uplift produced by 1.25 times the test pressure applied to the empty tank (with the as-built thicknesses)

c. The uplift produced by 1.5 times the calculated failure pressure, Pf, in accordance with *API 650* Section F.6 applied to the tank filled with the design liquid. The effective weight of the liquid should be limited to the inside projection of the ringwall from the tank shell. Friction between the soil and the ringwall may be included as resistance. If a footing is included in the ringwall design, the effective weight of the soil may be included.

6.4.2.4 Tanks with internal pressures in accordance with *API 650* Section F.7 (including F.7.5) should be anchored. The intent of the requirements of *API 650* Section F.7.5 is to size the foundation and anchorage of the tank such that any failure caused by an overpressure in the tank is forced to occur at the roof-to-shell junction of the tank instead of at the bottom-to-shell junction, thus preventing a product release. Providing emergency vents in accordance with *API 2000* or any other standard does not meet this intent or the requirements of *API 650* Section F.7.5.

6.4.2.5 When invoked, the requirements of *API 650* Section F.7.5 normally govern the design of a tank foundation.

6.4.3 Tanks on Grillage Beams

6.4.3.1 If tanks are required to have prompt leak detection systems, positive leak detection can be achieved by supporting the tanks on steel beams over a concrete pad. This arrangement provides a clear area where leaks can be seen. This type of foundation can generally be used for small tanks up to 20 ft in diameter.

6.4.3.2 Elevated tank foundations are more expensive than other types of small tank foundations. Therefore, the decision to support the tank on an elevated foundation should be justified by an economic comparison with other methods of secondary containment and leak detection. See *API 650* Appendix I for more information.

6.5 Berms and Gutters

6.5.1 Design requirements for the area outside the perimeters of large tanks should include 8-ft-wide minimum sloped berms to drain liquids away from the tanks and to facilitate maintenance and painting.

6.5.2 For tanks 20 ft diameter or less, berm widths should be 6 ft minimum. The slope of the berm should be 2% minimum, and the berm should be dressed to prevent erosion. Either a spray coating of suitable asphaltic binder material or a 2-inch minimum asphaltic concrete or other permanent paving material should be used. Compatibility with tank contents should be considered when selecting berm dressing.

6.6.3 Tank grades that are properly constructed require little maintenance except occasional oiling and clearing of gutters and drains.

6.6.4 Selecting a good berm dressing is particularly important for maintenance. Plant-mixed asphaltic concrete dressings are the most durable types, but many types of dressing using well-graded soils mixed with road oils have been successfully used.

6.5.5. The best type of dressing to use depends on the availability of material and cost. Asphaltic concrete is more expensive than oil-coated soils.

6.6 Grounding and Cathodic Protection

6.6.1 Metal tanks should be protected from static electricity, lightning, and stray currents. Refer to project electrical engineer and/or *NFPA 78* for additional requirements.

6.6.2 Tanks with metallic bottoms may require cathodic protection to prevent corrosion of the tank bottom.

6.6.3 The foundation design may require penetrations and/or sleeves to serve as a raceway for cabling for the grounding or cathodic protection systems. This requires coordination with the project electrical engineer.

6.7 Secondary Containment and Leak Detection Systems

6.7.1 Membrane Liner

6.7.1.1 For a ringwall foundation with secondary containment and leak detection, an HDPE membrane liner should be stretched over the compacted fill inside the ringwall and attached to the ringwall. For bottom replacement, the membrane should be placed on top of the old bottom.

6.7.1.2 The membrane should be placed after completion of the concrete ringwall, removal of the internal ring forms, and backfilling and compacting of the fill material inside the ringwall to the proper slope.

6.7.1.3 For cone-up tank bottom foundations, the membrane should be installed under the water-draw basin before the basin is poured.

6.7.1.4 For cone-down tank bottom foundations, the center sump and sump liner along with the telltale line from the sump liner to the standpipe outside the tank should be placed before installing the membrane.

6.7.1.5 For new foundations, the membrane liner should be impaled over the concrete ring foundation reinforcing bars extending vertically from the foundation. Alternatively, special embeds may be used to attach the liner to the ringwall.

6.7.1.6 For replacement tank bottoms, the membrane should be attached to the old bottom at the shell by adhesive/sealant. For cone-down tank bottoms, the old center sump should be cut out and replaced with a new sump and sump liner, and a telltale line should be run to a standpipe outside the tank.

6.7.1.7 The membrane liner should be as level, smooth, and free of wrinkles as practical before the sheets are extrusion-welded (or bonded) together. Extrusion welds (or lap joint adhesion) should be checked for bond and leakage. Bond can be checked with a dull ice pick, and leakage can be checked by a vacuum test similar to that used for welded steel plate seams.

6.7.1.8 For replacement tank bottoms, the membrane at the "rat holes" should be well sealed with adhesive/sealant. Rat holes are the cutouts in the old shell that allow leaks to drain from the grooves in the concrete pad and out to a gutter.

6.7.2 Concrete Pad and Grooves

6.7.2.1 A concrete pad (or spacer, for bottom replacements) is poured on top of the membrane liner.

6.7.2.2 If the pad is to be reinforced with polypropylene fiber or wire mesh, this material is placed on the membrane before the concrete is poured.

6.7.2.3 After the pour, grooves are cut into the pad to drain any liquid leaking from the tank to the outside where it can be seen.

6.7.2.4 Grooves in the concrete pad should be made by saw-cutting. As an option, the grooves can be "floated" while the concrete is fresh.

6.7.2.5 For cone-up tank bottom replacements, grooves should line up and extend to the "rat holes" cut in the existing shell. The last 12 to 15 inches should be chiseled.

6.7.2.6 For cone-down tank bottom replacements, grooves should stop 12 to 15 inches from the shell.

6.7.2.7 Together, the membrane liner and the grooves that are cut into the concrete pad are the secondary containment and leak detection system.

6.7.4 Telltale Pipes and Sumps

6.7.3.1 Telltale pipes carry the liquid from leaks away from the tank to where it can be seen.

6.7.3.2 For cone-down tank bottoms, telltale pipes should be checked for level and tested for leakage. The backfill should be tamped.

6.7.3.3 For installation of the telltale line for replacement tank bottom, the area under the concrete ringwall (or area under the shell) should be backfilled with concrete to avoid local settlement.

6.7.3.4 Telltale pipes may drain into the diked area or inspection standpipes.

6.7.3 Sumps

6.7.4.1 A sump should be placed at the specified elevation.

6.7.4.2 The sump should rest fully on well-compacted soil.

6.7.4.3 If the base under the sump has any tendency to shift or settle, an unformed, polypropylene fiber-reinforced 4-inch-thick pad should be installed and checked for elevation before the basin is installed.

6.8 Tank Settlement

6.8.1 General

6.8.1.1 Tanks are relatively flexible structures that tolerate a large amount of settlement without signs of distress. Tank settlement, however, has caused failures such as inoperative floating roofs, shell and roof buckling damage, leaks, and loss of tank contents. Foundation design, soil conditions, tank geometry and loading, and drainage, all can significantly affect settlement.

6.8.1.2 Large petroleum tanks are generally constructed on compacted soil foundations or on granular material, whereas smaller tanks are often built on concrete slabs. The settlements considered in this Practice pertain to large tanks (greater than 50 ft in diameter) because most large tanks are built on foundations having variable thickness, elasticity, and compressibility, and subsoil layers can vary enough to produce non-planar distortions if uniformly loaded. Nevertheless, the basic principles apply to all tanks, especially the principles pertaining to uniform settling and planar tilt.

When filled, tank contents will uniformly load the foundation beneath the tank as the result of hydrostatic pressure in a disk pattern. The tank edge, however, carries an increased load from the shell and roof weight and can experience loading effects such as twisting of the plates under the shell because of shell rotation. The tank edge is defined as the area of the tank composed of the tank shell, the roof supported by the shell, and the foundation directly beneath.

6.8.1.3 Most settlement problems occur in the part of the foundation that is under the outside edge of the tank. However, failures have occurred because of interior settling that went undetected in elevation readings. Settlement problems are assessed by taking elevation readings at the base of the tank.

6.8.1 Settlement Categories

6.8.2.1 Uniform Settlement

1. For soil conditions that are relatively uniform, soft, or compressible, a storage tank can slowly, but uniformly, sink downward, as shown in Appendix C, Figure C-1.
2. Uniform settling poses no significant problems; however, two important side effects should be considered:
   a. Water ingress can occur if a depression or water trap is formed around the tank periphery where it meets the soil. During a rain, moisture can accumulate under the tank bottom near the shell or chime region and corrode the tank bottom.
   b. Piping connected to the tank can eventually become over-stressed by the tank movement unless sufficient flexibility is designed into the piping system.
3. Elevations should be monitored at the base of the tank to assess the degree of uniform settlement.

6.8.2.2 Planar Tilt

1. In planar tilt mode, a tank tips as a rigid structure as shown in Appendix C, Figure C-2.
2. Planar tilt commonly accompanies uniform settlement.
3. Planar tilt can be assessed from an external tank inspection by taking elevation readings at the base of the tank.
4. The following should be considered if the tilt becomes severe.
   a. The typical human eye is sensitive to vertical lines. Therefore, with a relatively small angle of tilt, a tank can look out of plumb. The public or employees may question the safety of the tank and the operating and maintenance practices being used. A plumbness ratio limited to the tank diameter divided by 50 should provide an acceptable tank appearance.
   b. The tilt of the tank can result in an increase in hydrostatic head, as shown in Appendix C, Figure C-2. If the increased stress causes the shell to exceed the design-allowable stress, the liquid level should be lowered.
   c. Because the maximum liquid level is typically just beneath the roof or overflow, a reduction in the allowable liquid level may be needed to accommodate the planar tilt. The reduced

allowable liquid level results in reduced tank storage capacity.

d. If a tank tilts, the plan view will be an ellipse, as shown in Appendix C, Figure C-2. Because floating roof tanks have specific clearances and out-of-round tolerances for their rim seals to work properly, planar tilt can cause seal interferences. However, the amount of planar tilt has to be extreme before ovalizing could affect the operation of the seals.

6.8.2.3 Differential Shell Settlement

1. Differential settlement, alone or in combination with uniform settlement and planar tilt, results in a tank bottom that is no longer a planar structure.

2. This type of settlement can be assessed by taking elevation readings around the circumference of the tank shell where the bottom projects beyond the shell as discussed in *Simple Method Calculates Tank Shell Distortion*. The elevation readings can be plotted as shown in Appendix C, Figure C-3. If the bottom of the tank is planar, a cosine curve can be fitted through the measured points. If, however, there is differential edge settlement, a best-fit cosine curve can be fitted to these points. Refer to *API 653,* Appendix B, for a best-fit cosine procedure.

3. Differential shell settlement is more serious than uniform or planar tilt settlement because deflection of the structure on a local scale is involved, which produces high local stresses.

4. As shown in Appendix C, Figure C-4, differential settlement occurring in the tank bottom near the shell produces an out-of-round condition ovalizing at the top of tanks that are not restricted in movement (e.g., a floating roof tank). Because the seals in floating roof tanks have specific tolerance limits between the edge of the roof and the tank shell, ovalizing of the tank can interfere with the floating roof operation or destroy the seal itself.

   a. If the bending stiffness of a tank is much less than the extensional stiffness (thin wall structure), the theory of extensionless deformations may be used to compute the relationship between differential settlement and radial deformation at the top of the tank.

   b. With specific readings of settlement, the following finite difference equation may be used to estimate ovalizing:

$$r = \frac{DH}{2}\frac{N^2}{\pi^2}\Delta S_i$$

where:

r = radial shell displacement at top of tank

D = tank diameter

H = shell height at which radial displacements are calculated

N = number of stations or readings

i = station number of elevation reading taken at base of tank

ΔS_i = measured settlement at ith location

5. Non-planar, differential settlement can generate shell stress near the top of a tank that can result in buckling of the tank's upper shell courses. In the past, the amount of differential settlement allowed was determined by arbitrarily limiting the differential settlement to a constant, which was a ratio of the settlement to the span between consecutive settlement measurements.

   a. Appendix C, Figure C-5 shows how various structures, particularly buildings, are damaged if the slope represented by the deflection-to-span ratio exceeds various values.

   b. A commonly used equation for determining the settlement limit for a tank is

   $$\Delta S_a = \frac{L}{450}$$

   where:

   ΔS_a = allowable settlement

   L = length between settlement readings, ft

   However, local slopes limited to approximately L/450 to L/350 applied to tanks have proven to be conservative and resulted in tanks being releveled when further settlement could have been tolerated.

   c. The *API 653* formula uses a safety factor of two times. For carbon steel:

   $$\Delta S_a = 11\frac{F_y L^2}{2EH}$$

   F_y = Yield stress, psi

   E = Modulus of elasticity, psi

   H = Shell height, ft

6.8.2.4 Global Dishing

1. Global dishing occurs when an entire tank bottom settles relative to the shell either alone or in combination with other forms of settlement. The most common type of global dishing is when a

tank bottom forms a dished shape, as shown in Appendix C, Figure C-6.

2. The problems associated with general global dishing settlement are as follows:
   a. High stresses generated in the bottom plates and fillet welds
   b. Tensile stresses near the shell-to-bottom welds that can cause shell buckling
   c. Change in calibrated tank volumes (strapping charts and gauges)
   d. Change in the drainage of the tank bottom profile and puddling during attempts to empty tank
3. *Criteria for Settlement of Tanks* suggests maximum global dishing settlement values that range from D/50 to D/100, depending on foundation type, safety factor, or empirical data where D is the tank diameter in feet. For global dishing, these values appear to be reasonable.
4. The preceding recommendations are based upon the large deflection theory of circular flat plates with edges that are not free to move radially. However, if the difference in settlement between the center and the periphery of the tank is large indications are that the bottom membrane does move inward radially or the shell will be pulled in as shown in Appendix C, Figure C-6. From theoretical considerations, the difference in membrane stresses generated between a circular plate simply supported with a fixed edge and an edge that is free to move radially is a factor of about 3. This means that the stresses will be one-third as high for bottom plates that are free to slide as for those that are not. If a tank is loaded with liquid, the bottom plates are probably held in place more securely; therefore, it may not be a valid assumption to use the free edge condition.

6.8.2.5 Local Interior Settling

1. Local settling that occurs in the interior of tanks typically takes the form of depressions, as shown in Appendix C, Figure C-7. Local interior settling poses similar problems to global dishing, and the proposed methods of assigning a tolerance are again based upon the theory of large deflection. If the settling occurs near the tank wall, some of the methods of tolerance assigning include a relaxation to take into account the freedom of the plate near the shell to slide radially inward as the depression increases.
2. The tank fabrication process leads to buckles and bulges in the bottom plates. If the tank is filled with liquid, these tend to level out but often reappear when the liquid is removed. Most of the models currently proposed for developing settlement criteria do not take into account the initial waviness of the bottom.

3. Local interior settling is inevitable in compacted earth foundations because soil composition and thickness varies under the tank. Deformations are typically formed gradually, without sharp changes in slope, so that the bottom plates are adequately supported. Risk of failure from this type of settlement is minimal unless serious problems exist with the welding integrity.

4. If large voids form under the tank bottom, the bottom plates can lift off the soil completely, as shown in the lower sketch of Appendix C, Figure C-7. Although this is not typically a problem, a large void can lead to localized rippling effects.

6.8.2.6 Sloped Bottoms

1. The previous sections about settling apply to flat bottom tanks; however, three types of bottoms have intentionally built slopes:

   a. Single slope

   b. Cone-up

   c. Cone-down

   Because the design slope of these tank bottom types averages about 1 inch in 10 ft they can be considered flat bottoms and the previous sections cam be applied.

2. Cone-up bottoms, subject to general dish settlement, can tolerate more total settlement than either flat bottom, cone-down, or single-slope bottoms. As settling occurs, the cone-up bottom compresses and becomes flat. As the soil settles below the tank, the compressive stresses that were generated become relieved until the shell base becomes cone-down, approximately equal to the magnitude of the original cone-up condition. The lower sketch in Appendix C, Figure C-8 illustrates this settlement progression.

3. If the initial cone-up bottom slope is significant, if the settling is relatively uniform, and if the bottom is constructed with lap-welded joints, a phenomenon known as rippling can occur, usually during the hydrostatic test on newly constructed tanks. Because of the linear layout of bottom plates and the use of fillet welds, a crease or a fold can form, covering large parts of the diameter, as shown in Appendix C, Figure C-8. The ripples are typically unidirectional and occur in the long direction of the bottom plates. The crease can be very severe (a radius curvature of approximately 1 ft is not uncommon), indicating that yield stresses have been exceeded. The ripple can act as a stiffening beam and cause increased differential settlement and bottom failure.

6.8.2.7 Edge Settlement

1. Edge settlement occurs in the bottom plates near the shell, as shown in Appendix C, Figure C-9. Determining this condition from the exterior of the tank is difficult; however, seen from inside the tank, this is one of the most obvious forms of settling.

2. Most edge settlement occurs in tanks that have been built on grades or compressible soils. If the soil has not been compacted sufficiently or becomes soft when wet, the probability of edge settlement increases. This type of edge settlement is mainly due to increased loading on the foundation at the periphery from the weight of the shell and roof. Typically the foundation has not been extended far enough beyond the tank radius to prevent lateral squeezing of the foundation, as shown in *API 653*, Figure B-5.

3. Edge settling can occur locally in soft spots around the edge of the foundation; however, this type of settling typically involves a rather substantial portion of the tank.

4. Edge settlement is rarely seen in tanks that are constructed on reinforced concrete ringwall foundations but is more common if the tank is built on a crushed stone ringwall foundation.

5. The use of annular plates reduces edge settlement. However, the two fillet welds between the annular plate, shell, and the bottom plates can induce shrinkage stresses into the annular plate and cause upward bulges. Although not strictly edge settlement, these bulges may contribute to actual edge settlement by creating an initial slope in the annular plate, which in turn sets up residual stresses that can cause the tank bottom under the shell to apply greater downward pressure on the soil. The initial slope can be attributed to edge settlement if the actual cause is weld shrinkage Proper weld procedures, careful selection of the welding sequence for all welds in the bottom annular plate, and careful fit up of plates should minimize this problem.

6. *API 653* and other edge settlement criteria are based upon a model that is similar to the dishing models described in the previous sections. Because edge settlement involves substantial yielding of the bottom plates (apparent from the large deflections over short spans), any model that uses an allowable stress basis for limiting settlement is probably extremely conservative. A strain-limiting approach may be more appropriate.

7. *API 653*, Figures B-10 and B-11, respectively, can be used to determine the maximum allowable edge settlement for areas with bottom lap welds approximately parallel to the shell and perpendicular to the shell.

8. The maximum allowable edge settlement for areas with a lap weld at an arbitrary angle to the shell can be determined by using the following equation:

$$B = B_e - (B_e - B_{ew}) \sin \alpha$$

where B_{ew} and B_e are from *API 653*, Figures B-10 and B-11, respectively, and α is the angle of the weld to a tank centerline, as shown in *API 653*, Figure B-12.

6.8.3 Designing for Settlement

6.8.3.1 Depending on the degree and type of settlement expected as determined from similar installations in the area or from soil surveys, the following methods of designing for expected settlement should provide increasing effectiveness at correspondingly increased prices (listed from least to greatest):

a. Standard lap-welded bottom

b. Annular plates with lap-welded bottom

c. Butt-welded bottoms

6.8.3.2 Because the standard lap welded tank bottom is the most economical, a tendency exists to use this design even for locations where significant settlement is expected.

6.8.3.3 Unless needed for large differential settlement, the butt-welded tank bottom design is usually rejected on a cost/benefit basis.

Additional design features such as the following can provide further effectiveness:

a. Deeper levels of soil compaction

b. Crushed stone ringwalls

c. Reinforced concrete ringwalls

d. Slabs on ringwall foundations

6.8.4 Tank Releveling

6.8.4.1 General

1. Tank releveling is a common procedure for correcting the problems associated with excessive settlement such as elastically buckling shell plates, leakage in the bottom plates, excessive out-of-roundness, and high stresses.

2. If floating roof tank bottoms have experienced differential settlement, the roofs can bind and seals may become damaged or ineffective. Releveling can cause the tank to resume a round shape.

3. Tanks that have been inelastically buckled by settlement or tanks that have been constructed initially out-of-round are typically not improved by releveling.
4. Only reputable, experienced contractors who have carefully planned a proven and effective tank-releveling procedure should be used.
5. All tank releveling procedures should include the following elements:
    a. For floating roof tanks, the roof should be supported from the shell to prevent excessive stresses and the possibility of cracks occurring from differential movement. Appendix C, Figure C-10 shows one way of supporting the roof.
    b. If tank jacking methods are used, tanks should be jacked up approximately 10 ft high, to allow for bottom inspection, cleaning, removing contaminated soil where leakage has occurred, rebuilding the foundation if necessary, or coating from the underside.
    c. Support should be provided for fixed-roof supports to eliminate roof buckling and damage.
    d. The amount of differential jacking should be controlled to eliminate shell buckling or weld damage in the corner welds or in the bottom plates.
    e. Because large groups of workers are involved and mistakes could cause injuries or unanticipated costs, all work methods should be carefully reviewed for safety, environmental concerns, and proven practices. Those performing the work should have direct experience using the proposed methods.
    f. A releveled tank should usually be hydrostatically tested. However, hydrostatic testing may not be necessary for all releveled tanks, e.g., small tanks undergoing low shell stresses or those having had only limited jacking.
    g. Piping connected to tanks should be disconnected if releveling can produce excessive stresses that could cause equipment damage. Underground piping connections to the tank should be exposed for monitoring.

6.8.4.2 Shell Jacking

1. Shell jacking is a common releveling method in which lugs are welded to the shell near the base, as shown in Appendix C, Figure C-11. Typical lug spacing is about 15 ft.
2. Once the lugs are in place and a suitable jacking pad is set up, jacking proceeds around the tank circumference in small increments. Jacking in small increments prevents warping the bottom excessively out of plane. Shims are installed as the jacks

are moved around, and the tank can be raised to any desired elevation. The tank bottom will sag down but should not overstress the bottom welds if the welds are sound.

3. Typically specified tolerances should average about 1/4 inch of being level for any measured point on the tank perimeter at the bottom.
4. Jacking contractor responsibilities should include the following:
   a. Providing, designing, installing, and removing lugs
   b. Removing any weld arc strikes and grinding out remaining slag
   c. Recommending the prying load under each shimmed area to prevent foundation damage and settling. Shim spacing should be 3 ft.
   d. Proposing whether and how sand or grout should be applied to low points under the tank bottom
   e. If correcting an out-of-round tank, monitoring radial tolerances
   f. Providing complete written procedures for all work to be undertaken
5. If the jacking exposes a large unsupported area under the tank, applying a flowable grout or sand layer can provide a planar foundation for the tank bottom. However, miscellaneous injection of grout through holes cut through the bottom plates is typically ineffective and can interfere with releveling.
6. If the work is intended to correct out-of-round tanks, the radial tolerances and the effect of releveling on these tolerances should be frequently monitored. A minimum of eight equally spaced points at the top of the shell should be used for monitoring. Elevations and radial measurements should be made before and after the work.
7. A hydrostatic test should be conducted after the tank is releveled.

6.8.4.3 Under-the-Shell Releveling

1. The under-the-shell releveling method uses jacking under the bottom of the shell.
2. Therefore, the same procedures, specifications, precautions, and testing as covered in Section 6.8.4.2 should be observed. Small pits are excavated to hold the jack under the tank shell. Appendix C, Figure C-12 shows a typical jacking pit arrangement.
3. The principle objection to this method is that pits are excavated beneath the tank shell. In soil foundations, the excavations can cause a loss of compaction of 40% to 50%.

4. Because the spacing for shims and for jack points is greater than that for the shell-jacking method, higher soil stresses can be a problem while the work is in progress.

6.8.4.4 Pressure Grouting

1. The method of tank leveling by pressure grouting, often called sand pumping, is used to raise the elevation of low spots or settled areas or can be used to raise small or large areas where tank bottoms are low. Pressure grouting forces sand or grout under pressure into a low area to stabilize the bottom plates.
2. If the involved areas are small and numerous, pressure grouting is generally ineffective because the grout mixture flows through the areas of least resistance and can lift the plates even further. This method could also cause the tank bottom to rest on points rather than lie uniformly.
3. Pressure grouting has been effectively used to level areas under fixed roof supports.
4. The grouting contractor should provide written, step-by-step procedures for all work to be undertaken.
5. Before cutting the tank bottom to inject grout, precautions should be taken to handle the possible existence of flammable liquids or toxic substances that could have been previously stored or leaked.

7. Tank Hydrotest

7.1 Tanks should be hydrotested to ensure the integrity of the tank, to prevent differential settlement, and to reduce short-term settlement in compressible soils.

7.2 The hydrotest should be completed before connecting piping.

7.3 Hydrotesting should be in accordance with *API 650*.

7.4 A minimum of eight, equally spaced reference points should be established around the shell of the tank. Elevations of the reference points should be taken before the start of the hydrotest and at daily intervals during filling and hold periods.

7.5 An elevation benchmark should be located at least two tank diameters away from the tank foundation and noted on hydrotest documentation. The benchmark should be a permanent fixture.

7.6 A tank should be filled at the rate to be recommended by the geotechnical engineer.

7.7 Tank filling should be suspended if settlement exceeds the amount anticipated by the geotechnical engineer or tank manufacturer.

7.8 After tank filling, the water level in the tank should be maintained until the tank settlements cease or until the settlement rate becomes less than the value recommended by the geotechnical engineer.

7.9 Documentation of all settlement surveys should be maintained by the owner and engineer of record and used for evaluation of future tank settlement problems.

7.10 See *PIP VESTA002*, Section 5.3.6(D), for more information regarding tank hydrotesting.

Appendix A - Ringwall Foundation Design Examples

Example 1 – El Segundo, California

Given: Fixed Roof Tank - 120-Ft Diameter x 40-Ft Cone Roof Tank

Tank Geometry:

R = 60 ft

H = 39 ft (maximum product level)

Product:

Contents: crude oil with specific gravity = 0.9

Internal pressure: atmospheric

Environmental Loads:

Wind: 85 mph, exposure C, I = 1.0 in accordance with *ASCE 7-02*

Seismic: *ASCE 7-02*

$S_S = 1.758$, $S_1 = 0.538$, Site Class D, I = 1.0

$S_{DS} = 1.172$, $S_{D1} = 0.538$, $A_v = 0.188$

Snow: none

Live load: 25 psf in accordance with *API 650*

Foundation Data:

fy = 60 ksi (rebar)

f'c = 3 ksi (concrete)

L = 0.67 ft (greater than minimum value of 5 inches)

h = 3 ft

e = 1 ft

k = 0.3 (granular backfill used)

γ_s = 100 pcf (backfill)

γ_c = 150 pcf (concrete)

q^a (normal operating) = 2,500 psf (net)

q^a (operating with wind) = 3,333 psf (net)

q^a (operating with seismic) = 3,333 psf (net)

q^a (hydrotest) = 3,000 psf (net)

Foundation Loads at Tank Shell (Service Level):

W_P = 2,190 psf (operating)

W_P = 2,434 psf (hydrotest)

D_e = 834 plf (empty dead load of tank)

E = 4,547 plf (seismic load from the overturning moment determined from the horizontal seismic acceleration—already reduced by 0.7 load factor)

W = 93 plf (wind)

L_R = 469 plf (roof live load)

S = 0 plf (roof snow load)

Determine Ringwall Dimensions:

Determine applicable load combinations (LC) from Table 9 of *PIP STC01015*:

LC	Description
2	Test Weight + Test Pressure
4	Operating Weight + Wind
5	Operating Weight + Live
8	Operating Weight + Seismic

$$b_{LC\,2} = \frac{834 + 2434(0.67)}{3000 + (3-1)100 - 3(150)} = 0.90 \text{ ft}$$

$$b_{LC\,4} = \frac{(834 + 93) + 2190(0.67)}{3333 + (3-1)100 - 3(150)} = 0.78 \text{ ft}$$

$$b_{LC\,5} = \frac{(834 + 469) + 2190(0.67)}{2500 + (3-1)100 - 3(150)} = 1.23 \text{ ft}$$

$$b_{LC\,8} = \frac{(834(1 + 0.4(0.188)) + 4547) + 2190(1 + 0.4(0.188))(0.67)}{3333 + (3-1)100 - 3(150)(1+0.4(0.188))} = 2.30 \text{ ft}$$

Note the use of the vertical seismic acceleration modification term $(1 + 0.4A_v)$ in Load Combination 8.

∴ use b = 2 ft - 6 inches.

Determine Hoop Steel

Determine applicable LC from *PIP STC01015* Table 9:

LC	Description
2	Test Weight + Test Pressure
5	Operating Weight + Live

By inspection, Load Combination 2 will control.

$$T_h = 60\,(3)(0.3)\left(2434 + \frac{100(3)}{2}\right) = 139{,}536 \text{ lb}$$

Soil load factor = 1.6 $\phi = 0.9$ for tension.

$$A_{S\ HOOP} = \frac{1.6(139{,}536)}{0.9(60{,}000)} = 4.13 \text{ inches}^2$$

Determine Twist Steel

Determine factored twist moment using *PIP STE03020* Figure 5. By inspection, Load Combination 8 controls because of large seismic load.

ACI 318 dead load factor = 1.2

ACI 318 fluid load factor = 1.2

Seismic load factor = 1.4* (Required to convert ASD seismic load to strength level.)

Load Description	Load (plf)	Load Factor	x_bar (ft)	M_T (ft-lb/ft)
Dead Load	834	1.2	0.58	580.5
Product	2190*0.67	1.2	0.92	1619.9
Seismic (from OTM)	4547	1.4	0.58	3692.2
Vertical Seismic Acceleration Applied to D_e	834*0.4*0.188	1.4	0.58	50.8
Vertical Seismic Acceleration Applied to W_P	2190*0.67*0.4*0.188	1.4	0.92	142.12
			Total	6085.5

$$M_u = 6085.1\left(60 - 0.67 + \frac{2.5}{2}\right)12 = 4{,}423{,}915 \text{ inch - lb}$$

b = 30 inches d = 36 – 3 – 0.5 – 0.5 = 32 inches

$$A_{S\ TWIST} = \rho\, b\, d$$

$$\rho = \frac{0.85 f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85 f_c'}}\right)$$

$$R_n = \frac{M_u}{\phi\, b\, d^2} \qquad \phi = 0.9$$

$$\rho_{\min} = \frac{200}{f_y}$$ *ACI 318* Section 10.5.2

$$\rho_{\max} = \frac{4}{3}\rho$$ *ACI 318* Section 10.5.3

$$R_n = \frac{4{,}423{,}915}{0.9(30)(32^2)} = 160.0$$

$$\rho = \frac{0.85(3000)}{60000}\left(1 - \sqrt{1 - \frac{2(160.0)}{0.85(3000)}}\right) = 0.0028$$

$$\rho_{\min} = \frac{200}{60000} = 0.0033 \qquad \text{controls}$$

$$\rho_{\max} = \frac{4}{3}(0.0028) = 0.0037$$

$$A_{S\ TWIST} = 0.0033(30)(32) = 3.17\ \text{inches}^2$$

Determine Minimum Temperature and Shrinkage Steel

Vertical = 0.0012 (15) (12) = 0.216 inch^2/ft each face

∴ use 360 #4 stirrups at approximately 13 inches on R = 61 ft - 7 inches.

Horizontal = 0.0025 (12) (36) = 1.08 inches^2/ft each face

Total horizontal required = 2 (1.08) = 2.16 inches^2 < hoop + twist = 7.30 inches^2

Final Design – Example 1

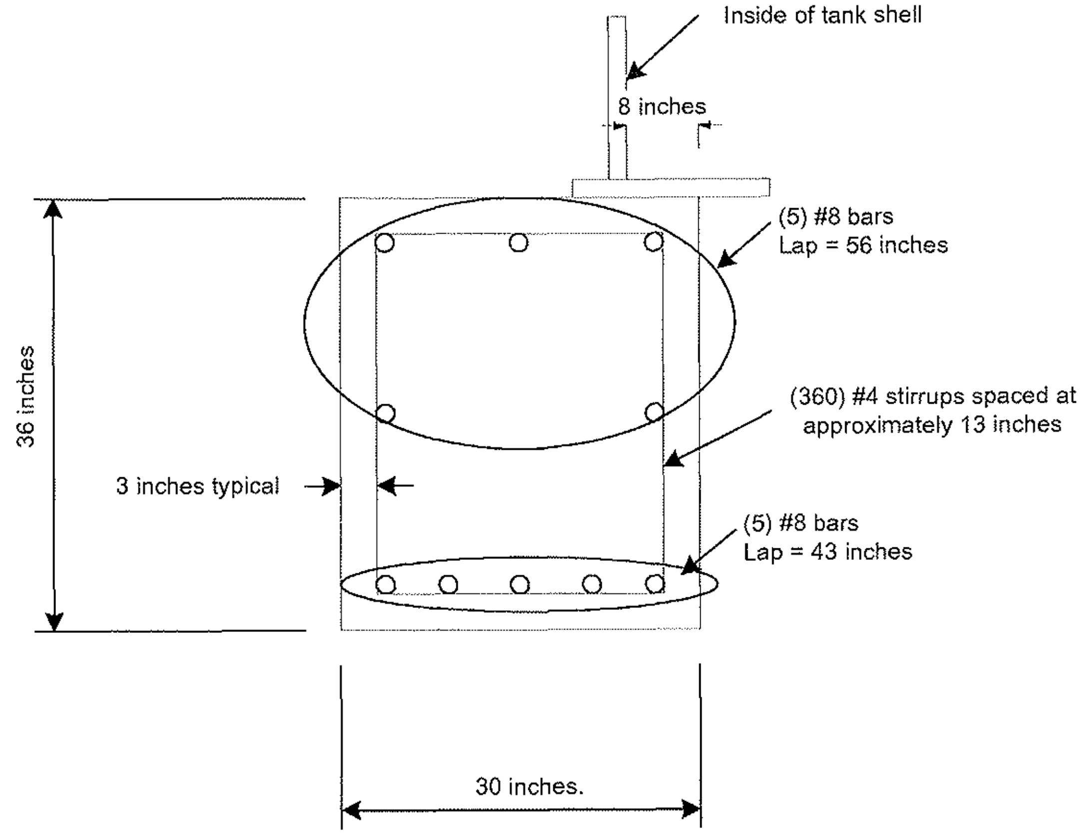


Figure A-1. Final Design - Example 1

Comments:

1. Stirrups are used to provide support for bottom and top steel.
2. One additional #8 bar is provided as top steel to reduce rebar spacing below 18 inches.
3. A_s provided = 7.9 inches^2 > 7.30 inches^2 A_s required.
4. Lap splices are Class B in accordance with *ACI 318*.

Example 2 – Corpus Christi, Texas

Given: Fixed Roof Tank - 120 Ft Diameter x 40 Ft CRT

Tank Geometry:

R = 60 ft

H = 39 ft (maximum product level)

Product:

Contents: crude oil with specific gravity = 0.9

Internal pressure: atmospheric

Environmental Loads:

Wind: 135 mph, Exposure. C, I = 1.0 in accordance with *ASCE 7*-02

Seismic: SDC A – none to consider in accordance with *API 650*

Snow: none

Live load: 25 psf in accordance with *API 650*

Foundation Data:

fy = 60 ksi (rebar)

f'c = 3 ksi (concrete)

L = 0.67 ft (Same as Example 1 for comparison purposes. Tank shell can be placed on ringwall centerline [L = 0.75 ft] without affecting the Example 2 design.)

h = 5 ft

e = 1 ft

k = 0.3 (granular backfill used)

γ_s = 100 pcf (backfill)

γ_c = 150 pcf (concrete)

q^a (normal operating) = 2,500 psf (net)

q^a (operating with wind) = 3,333 psf (net)

q^a (operating with seismic) = 3,333 psf (net)

q^a (hydrotest) = 3,000 psf (net)

Foundation Loads at Tank Shell (Service Level):

W_P = 2,190 psf (operating)

W_P = 2,434 psf (hydrotest)

D_e = 834 plf (empty dead load of tank)

E = 0

W = 178 plf (wind)

$L_R = 469$ plf (roof live load)

$S = 0$ plf (roof snow load)

Determine Ringwall Dimensions:

Determine applicable load combinations (LC) from *PIP STC01015* Table 9:

LC	Description
2	Test Weight + Test Pressure
4	Operating Weight + Wind
5	Operating Weight + Live

$$b_{LC\,2} = \frac{834 + 2434(0.67)}{3000 + (5-1)100 - 5(150)} = 0.93 \text{ ft}$$

$$b_{LC\,4} = \frac{(834 + 178) + 2190(0.67)}{3333 + (5-1)100 - 5(150)} = 0.85 \text{ ft}$$

$$b_{LC\,5} = \frac{(834 + 469) + 2190(0.67)}{2500 + (5-1)100 - 5(150)} = 1.29 \text{ ft}$$

∴ use b = 1 ft – 6 inches.

Determine Hoop Steel

Determine applicable load combinations (LC) from *PIP STC01015* Table 9:

LC	Description
2	Test Weight + Test Pressure
5	Operating Weight + Live

By inspection, Load Combination 2 will control.

$$T_h = 60\ (5)(0.3)\left(2434 + \frac{100(5)}{2}\right) = 241{,}560 \text{ lb}$$

Soil load factor = 1.6 $\phi = 0.9$ for tension.

$$A_{S\,HOOP} = \frac{1.6(241{,}560)}{0.9(60{,}000)} = 7.16 \text{ inches}^2$$

Determine Twist Steel

Determine factored twist moment using *PIP STE03020* Figure 5. By inspection, Load Combination 5 controls because of large seismic load.

ACI 318 dead load factor = 1.2

ACI 318 fluid load factor = 1.2

ACI 318 live load factor = 1.6

Load Description	Load (plf)	Load Factor	x_bar (ft)	M_T (ft-lb/ft)
Dead Load	834	1.2	0.08	80.1
Product	2190*0.67	1.2	0.42	730
Live Load	469	1.6	0.08	60.0
			Total	870.1

$$M_u = 870.1\left(60 - 0.67 + \frac{1.5}{2}\right)12 = 627{,}342 \text{ inch - lb}$$

$b = 18$ inches $\qquad d = 60 - 3 - 1.128/2 = 56.4$ inches

$$A_{S\,TWIST} = \rho\, b\, d$$

$$\rho = \frac{0.85 f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85 f_c'}}\right)$$

$$R_n = \frac{M_u}{\phi\, b\, d^2} \qquad \phi = 0.9$$

$$\rho_{min} = \frac{200}{f_y} \quad \textit{ACI 318} \text{ Section 10.5.2}$$

$$\rho_{max} = \frac{4}{3}\rho \quad \textit{ACI 318} \text{ Section 10.5.3}$$

$$R_n = \frac{627{,}342}{0.9(18)(56.4^2)} = 12.17$$

$$\rho = \frac{0.85(3000)}{60000}\left(1 - \sqrt{1 - \frac{2(12.17)}{0.85(3000)}}\right) = 0.0002$$

$$\rho_{min} = \frac{200}{60000} = 0.0033$$

$$\rho_{max} = \frac{4}{3}(0.000049) = 0.00027 \qquad \text{governs}$$

$$A_{S\,TWIST} = 0.00027(18)(57) = 0.28 \text{ inch}^2$$

Determine Minimum Temperature and Shrinkage Steel

Vertical = 0.0012 (18 / 2) (12) = 0.130 inch^2 / ft each face

∴ use (256) #4 bars in each face (512 total) at approximately 18 inches.

Horizontal = 0.0025 (18 / 2) (60) = 1.35 inches^2 / ft each face

Total horizontal required = 2 (1.350) = 2.70 inches^2 < hoop + twist = 7.44 inches^2

Final Design – Example 2

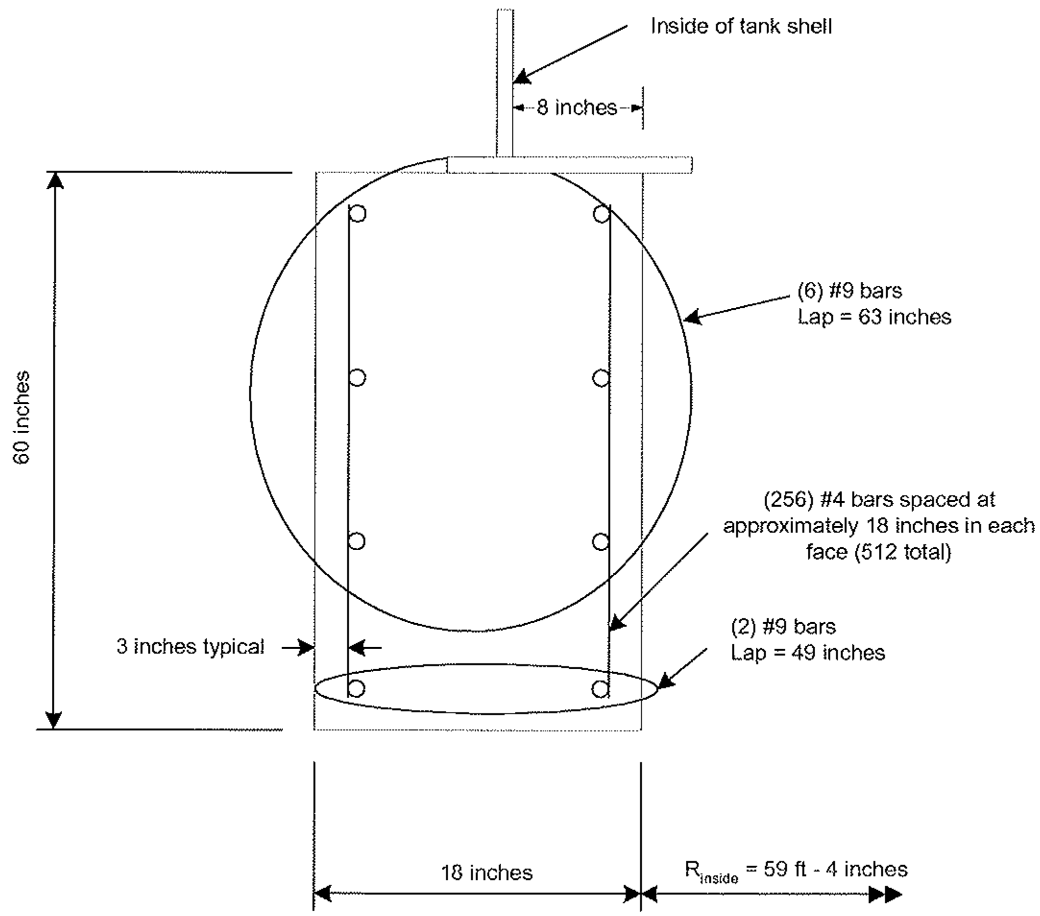


Figure A-2. Final Design - Example 2

Comments:

1. Twist is not a significant factor in this design example.
2. A_s provided = 8.0 inches^2 > 7.23 inches^2 A_s required.
3. Lap splices are Class B in accordance with *ACI 318*.

Appendix B – Hot Tanks

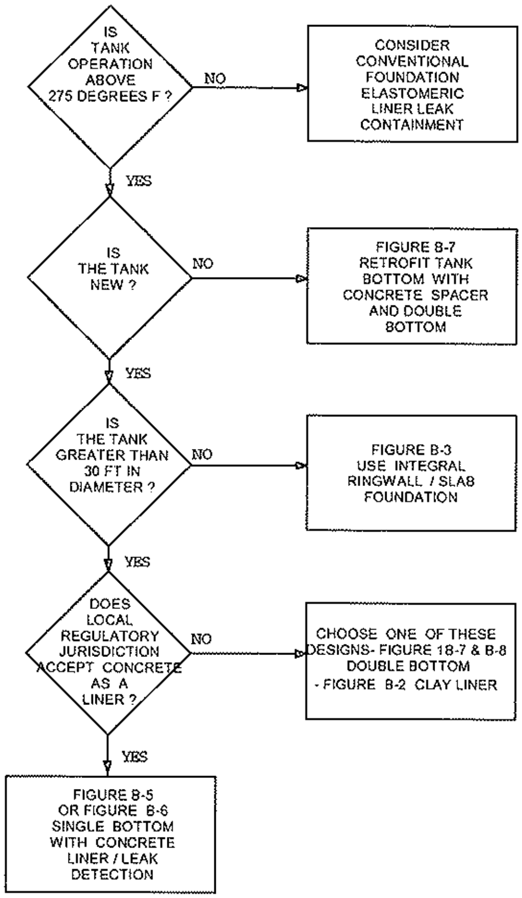


Figure B-1. Selection of Leak Detection/Leak Containment for Hot Tanks

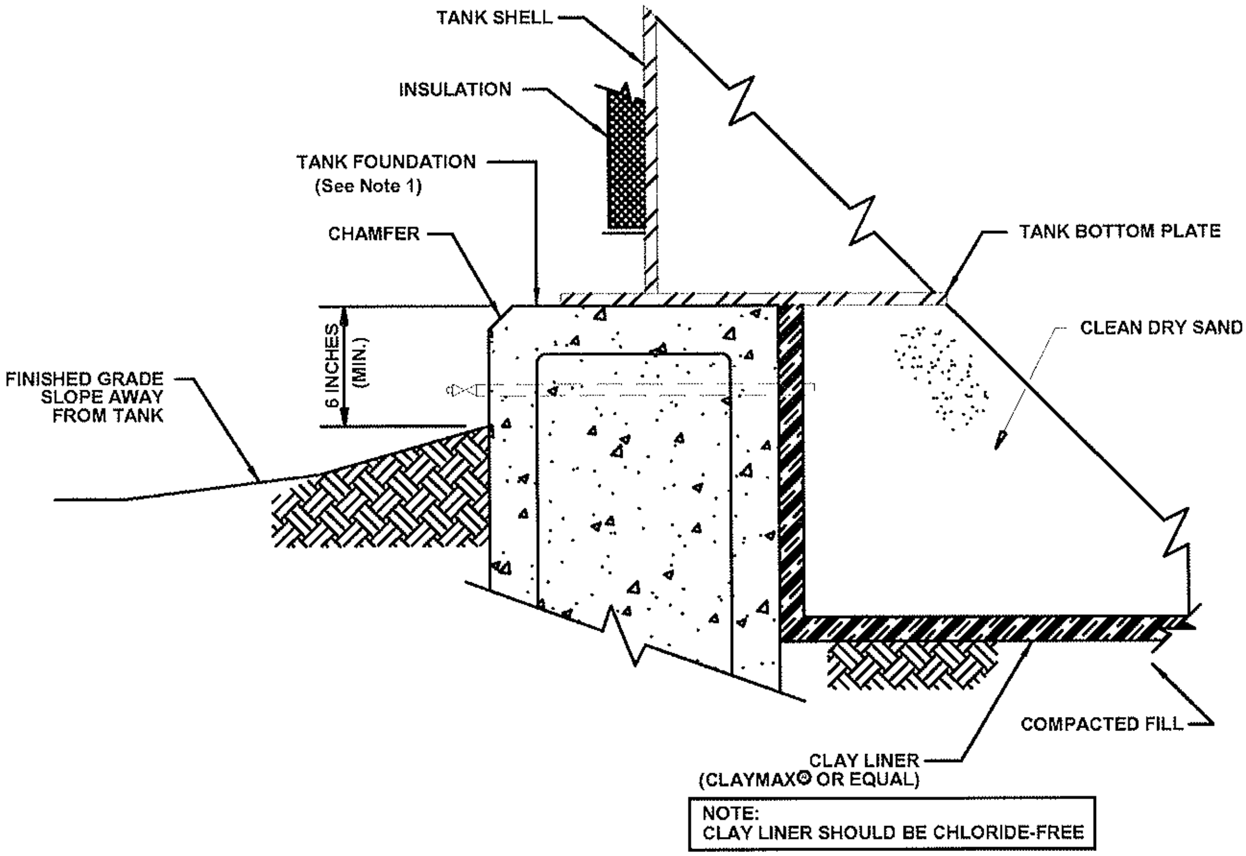


Figure B-2. High-Temperature Tank Foundation with Leak Detection and Containment Using Clay Liner

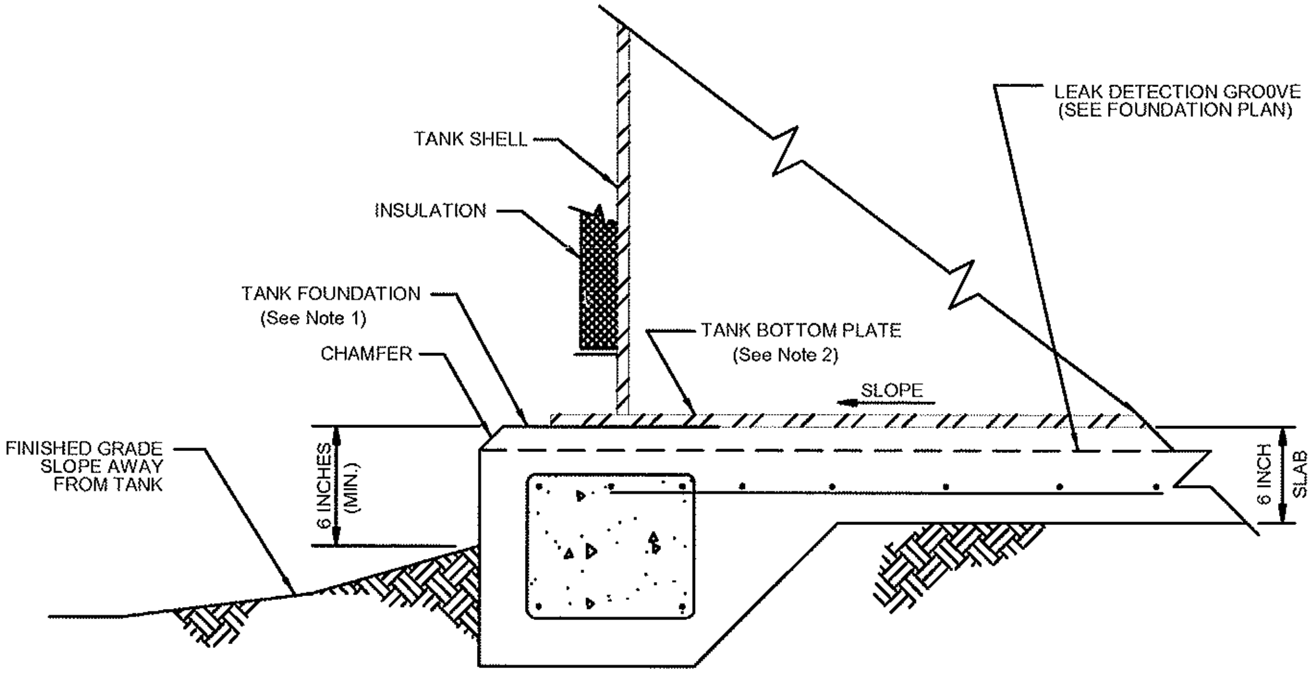
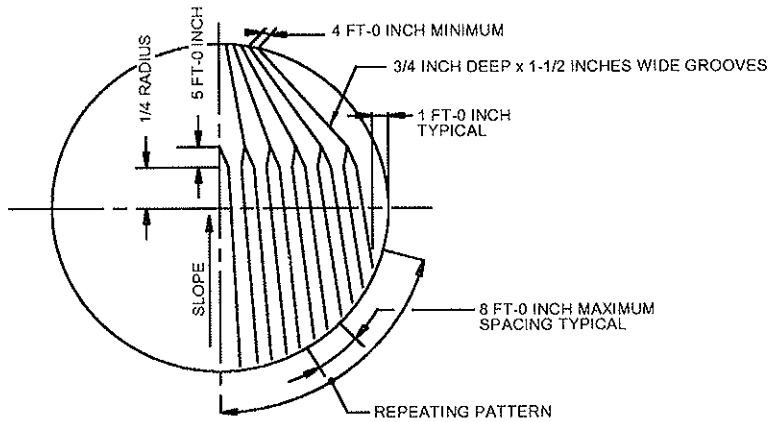
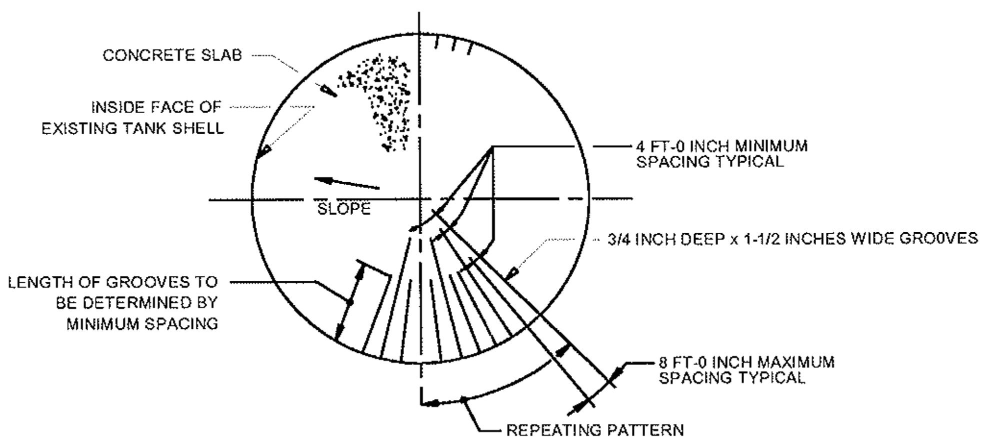


Figure B-3. High-Temperature Tank Foundation with Leak Detection for Small Tanks – Tanks < 30 Ft in Diameter



SINGLE SLOPE CONFIGURATION

(SEE NOTE 3)
GROOVING IS TYPICAL FOR
FIGURES B-3, B-5, B-7, B-8



CONE-UP CONFIGURATION

(SEE NOTE 3)
GROOVING IS TYPICAL FOR
FIGURES B-3, B-5, B-7, B-8

Figure B-4. Hot Tank Leak Detection Foundations

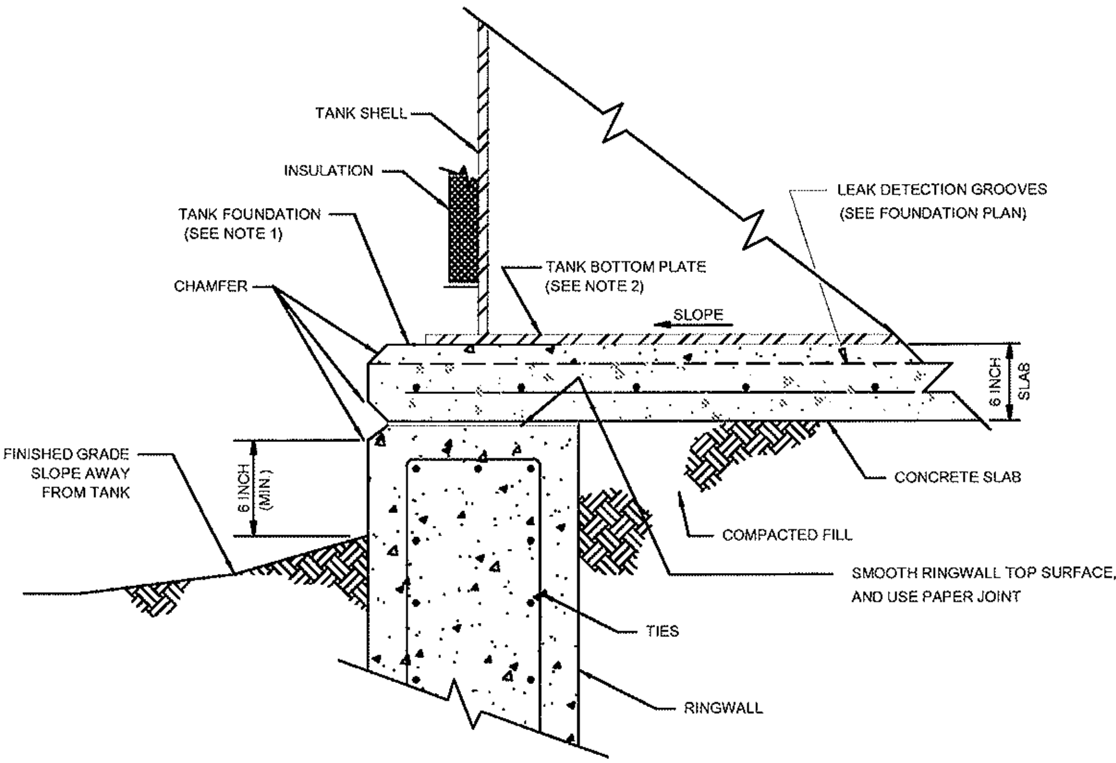


Figure B-5. High-Temperature Tank Foundation with Leak Detection – Tanks >20 Ft in Diameter

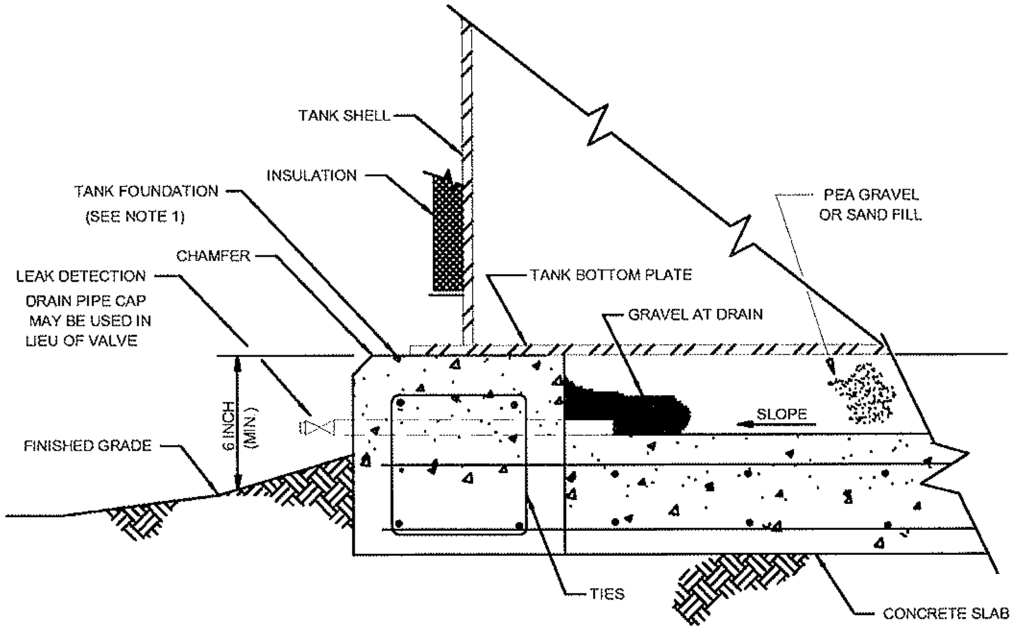


Figure B-6. High-Temperature Tank Foundation with Leak Detection and Leak Containment

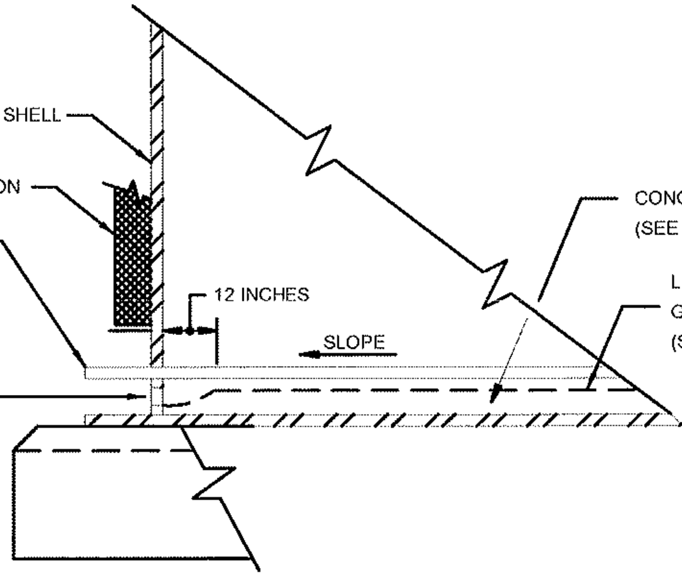


Figure B-7. Retrofit Existing with New Bottom to Include Leak Detection

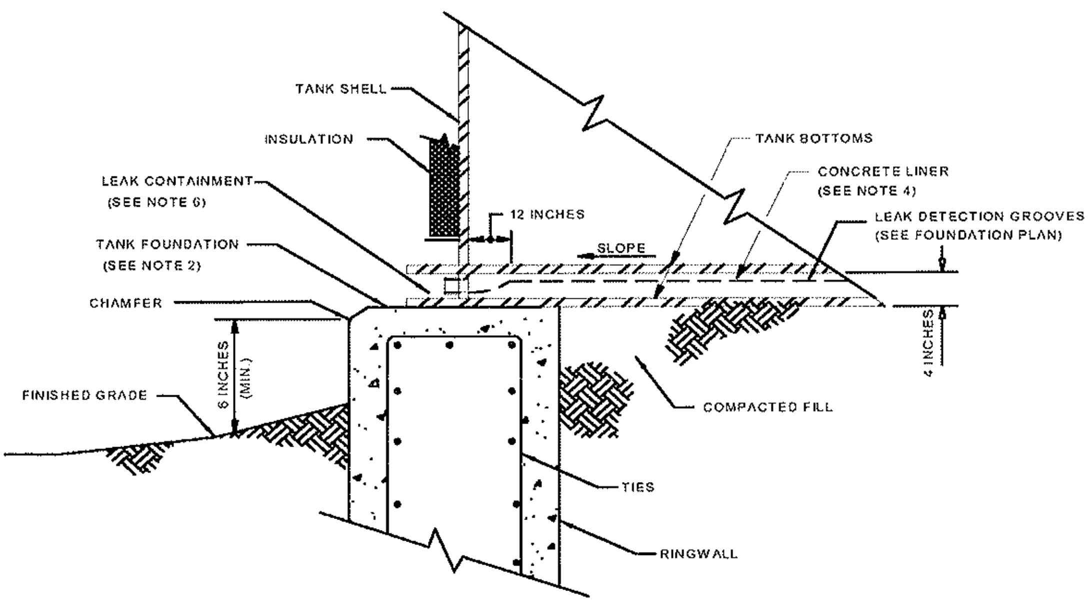


Figure B-8. High-Temperature Tank Foundation with Leak Detection and Leak Containment

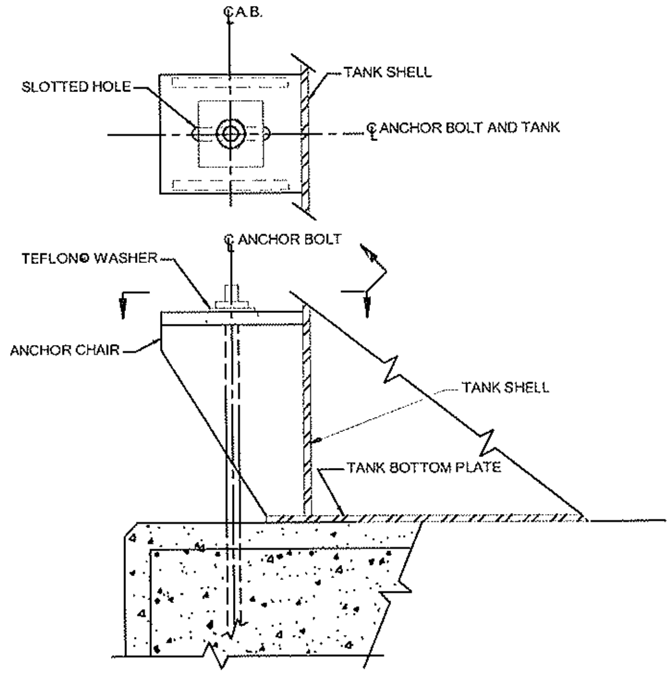
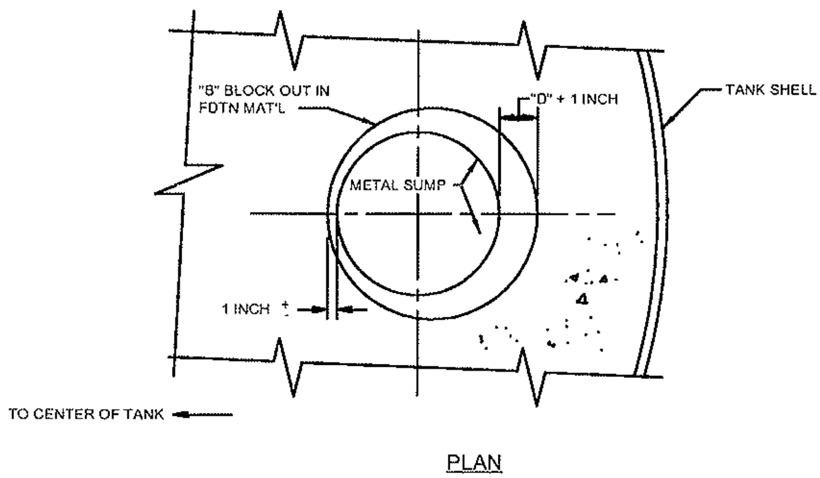


Figure B-9. High-Temperature Tank Anchor Detail



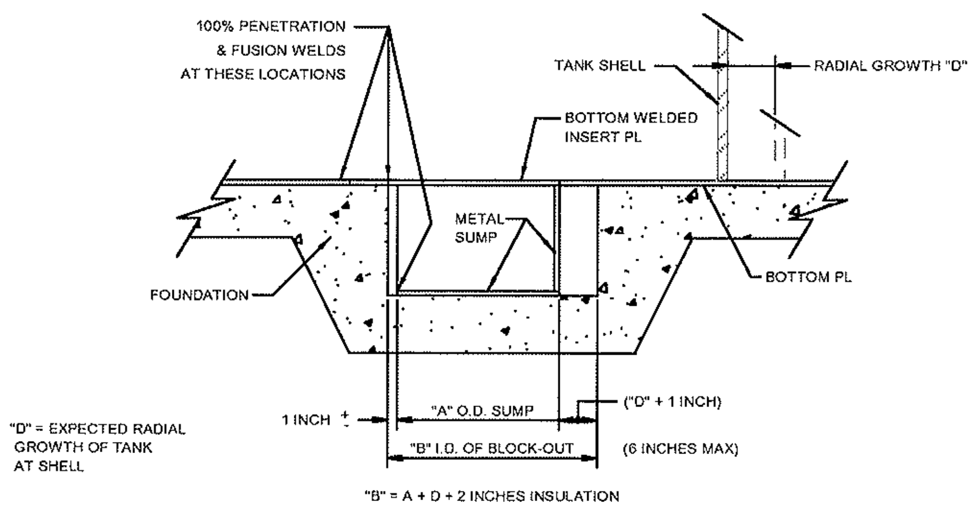


Figure B-10. Hot Tank Sumps

Notes for Figures

1. Edge of concrete surface should slope away from the tank to prevent water infiltration under tank bottom.
2. Foundation should be cone-up or single slope. Slope should not be less than 2 inches in 10 ft.
3. For small tanks, the 4-ft - 0-inch minimum spacing between grooves should be reduced.
4. Spacer reinforcement should be *ASTM A185*, 6-inch x 6-inch - W1.4 x W1.4 welded wire reinforcement. Splices should have a 6-inch minimum lap.
5. Where grooves are at the edge of the tank, existing steel should be notched for leak detection.
6. Where grooves are at the edge of the tank, a coupling should be installed in the tank shell.

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Appendix C – Tank Settlement

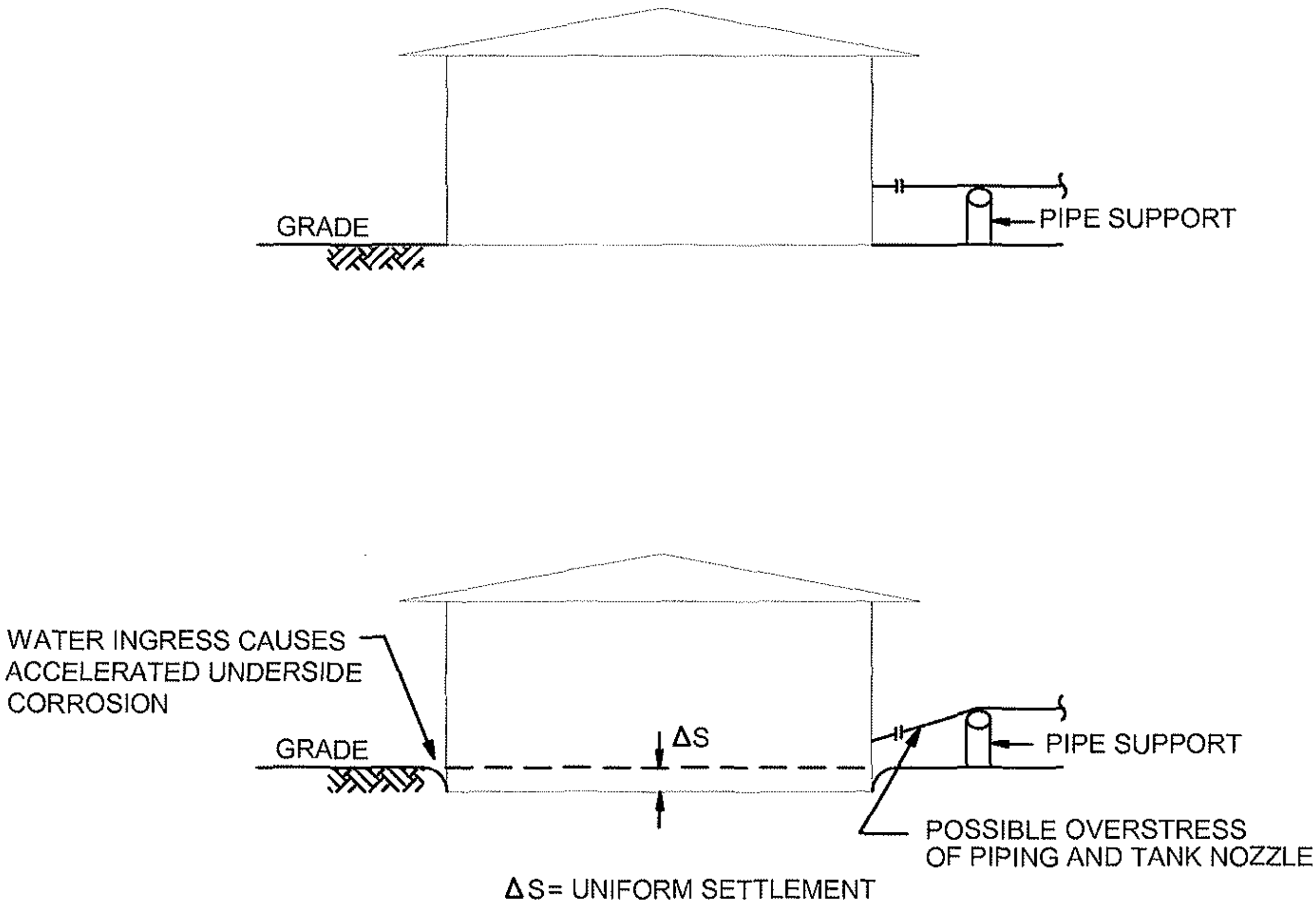

Figure C-1. Uniform Settlement

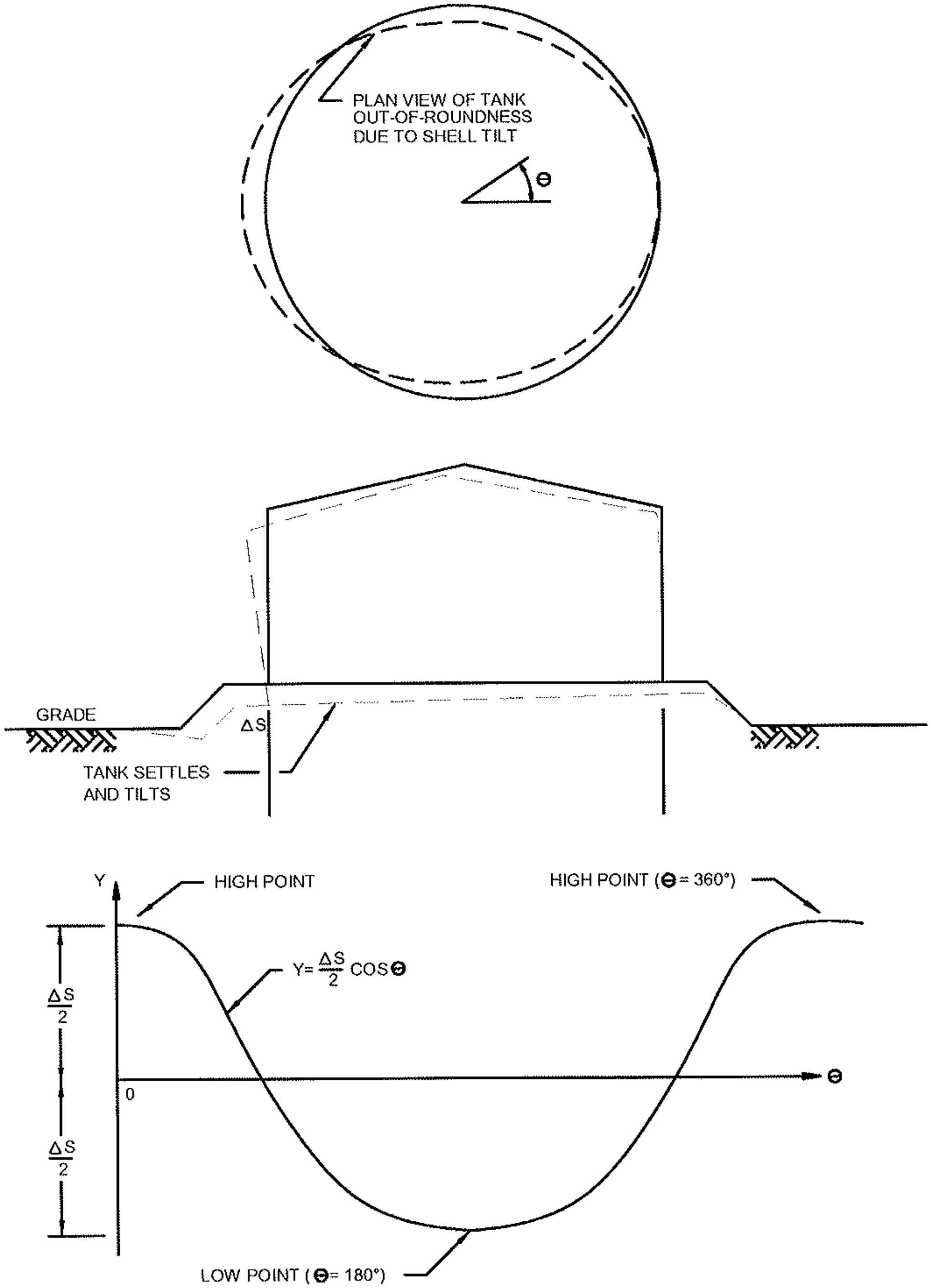

Figure C-2. Planar Tilt Settlement

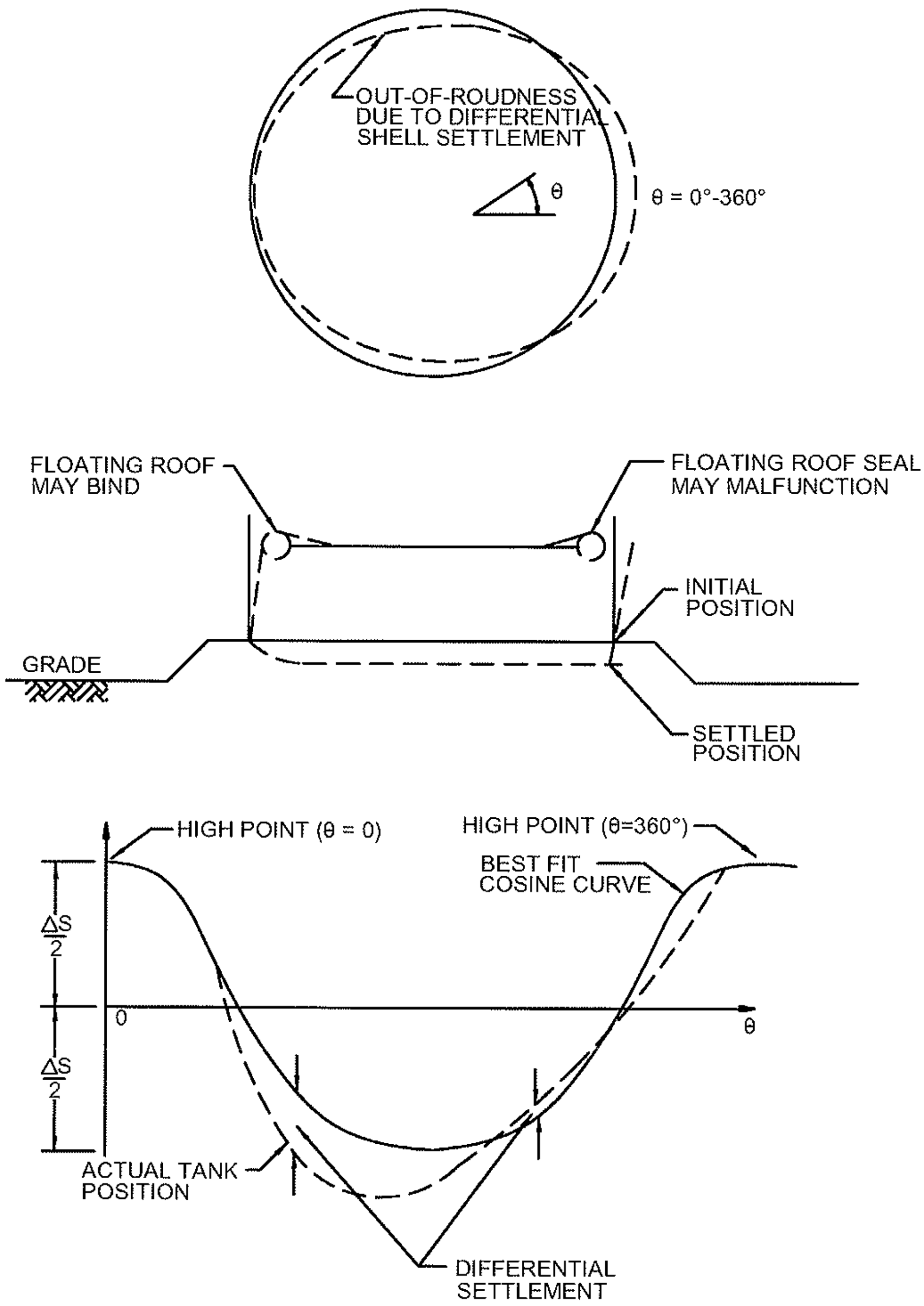

Figure C-3. Differential Tank Settlement

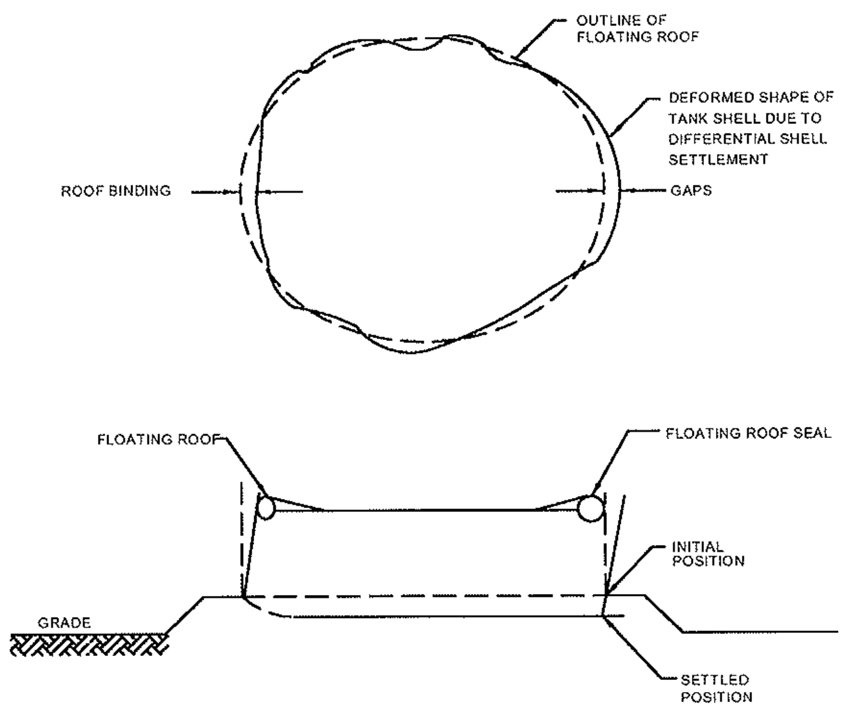

Figure C-4. Problems Resulting from Shell Out-of-Roundness Caused by Non-Uniform Settlement

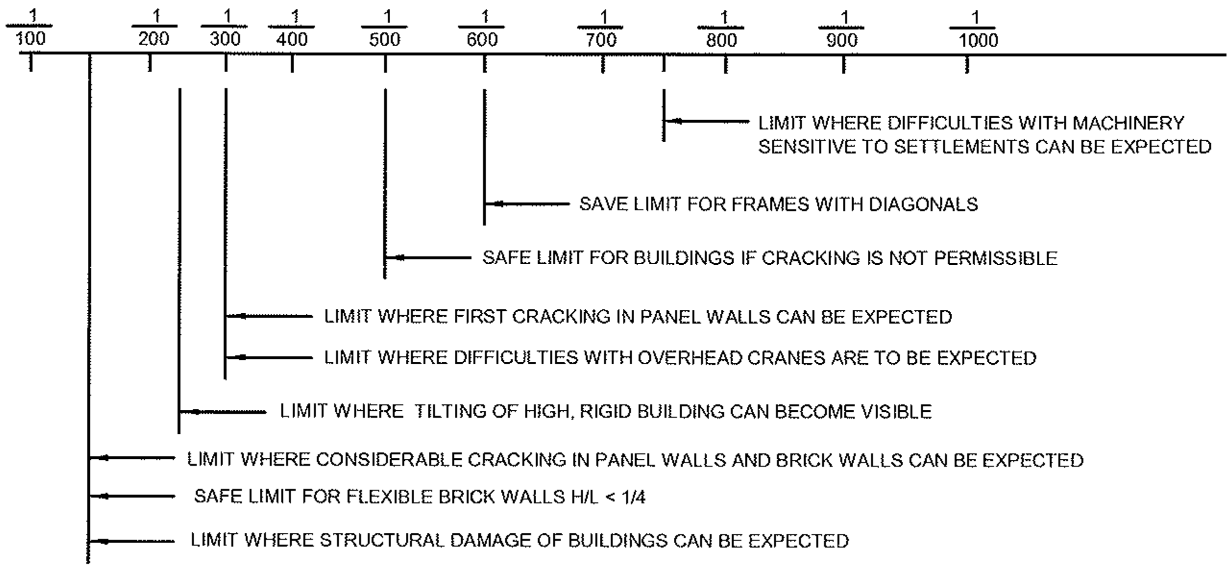

Figure C-5. Limiting Angular Distortion

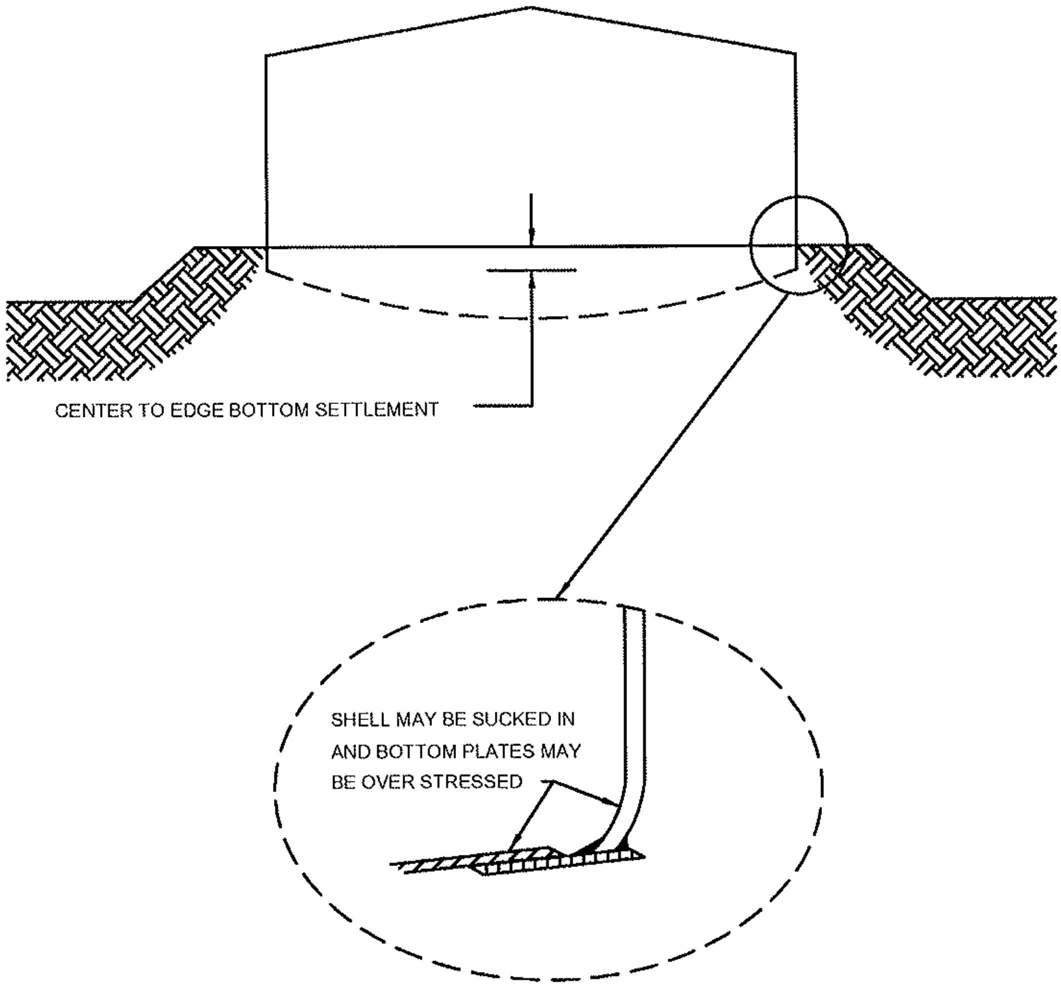

Figure C-6. Dish Settling

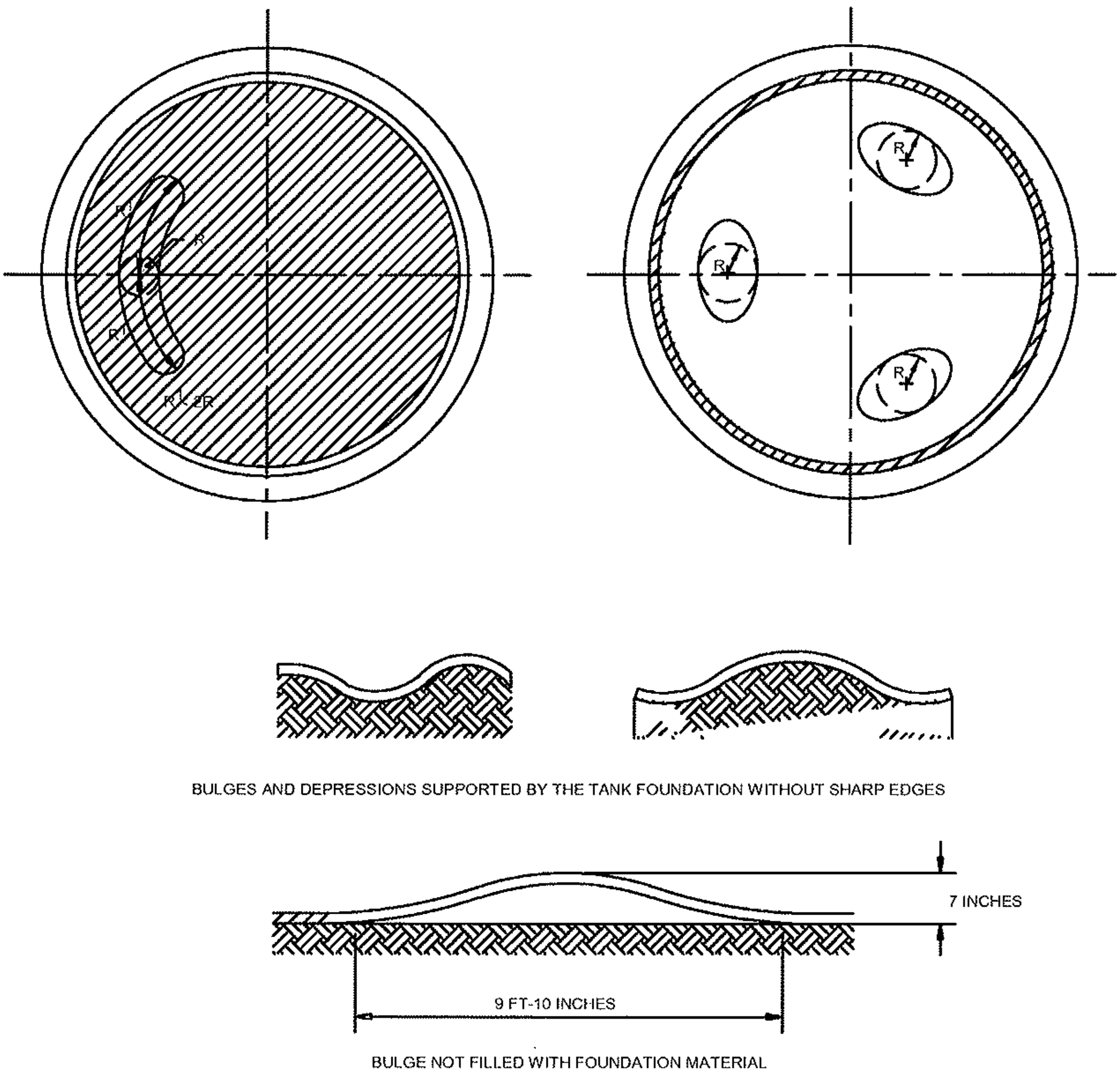

Figure C-7. Bottom Settlement

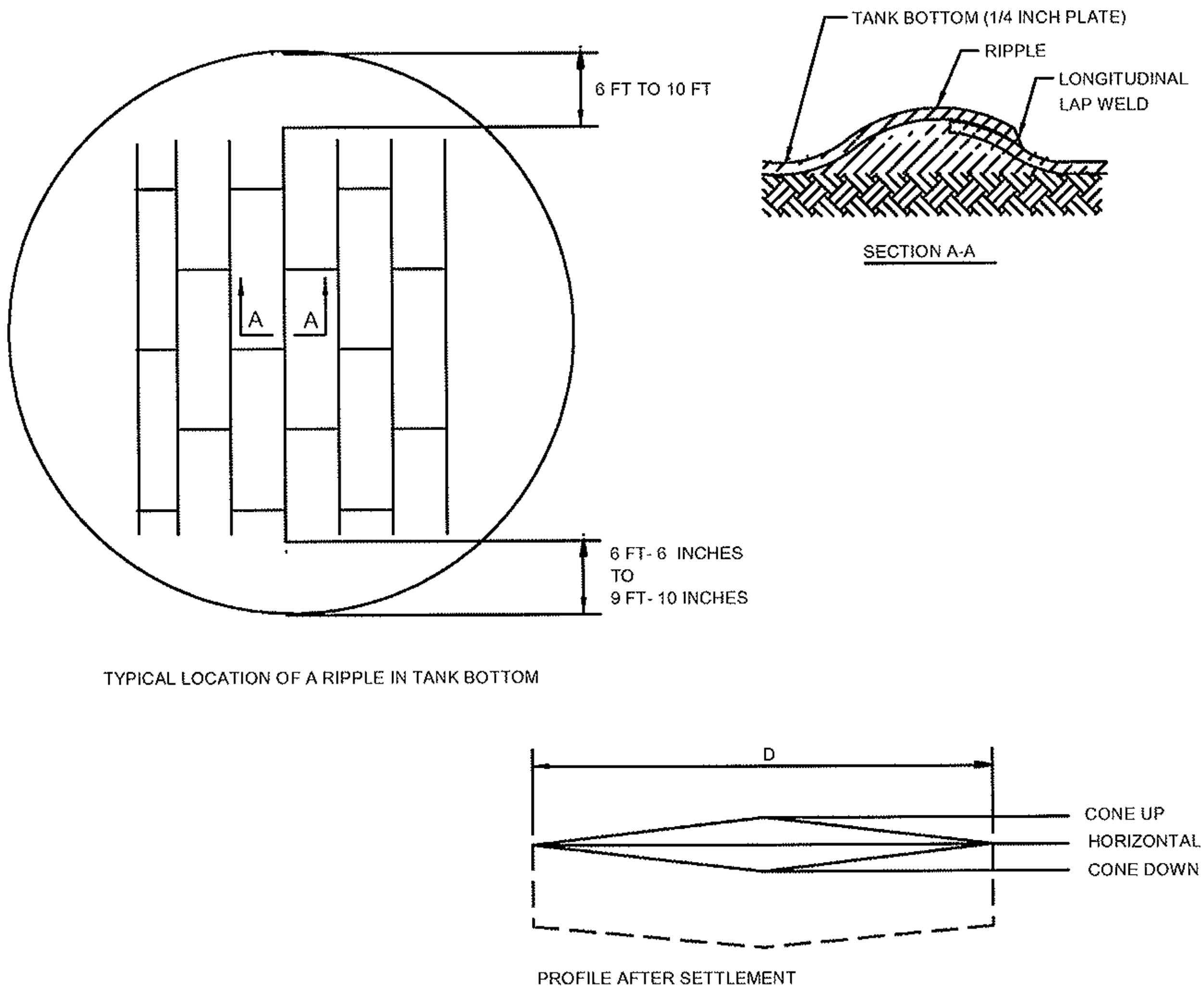

Figure C-8. Tank Bottom Ripples

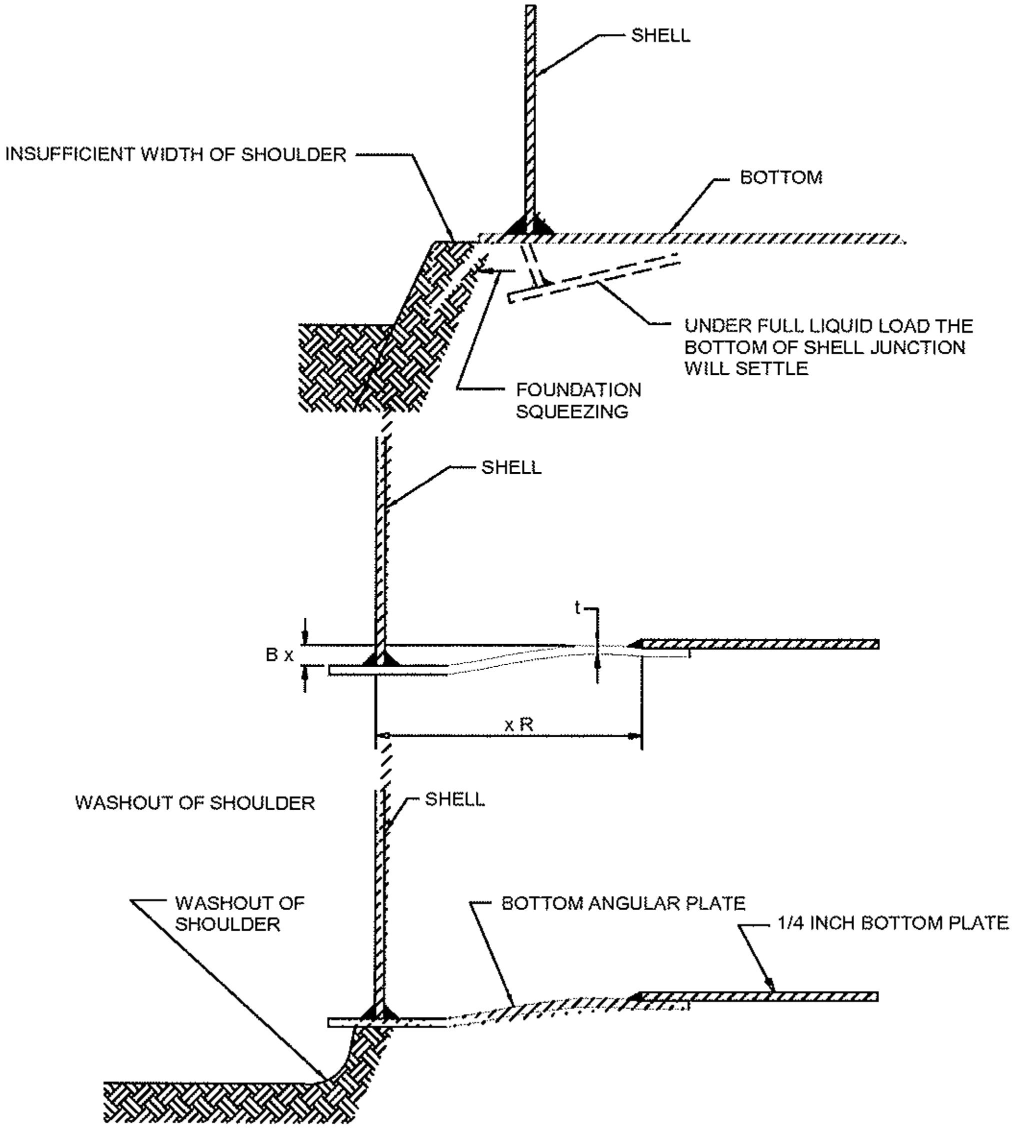

Figure C-9. Edge Settlement

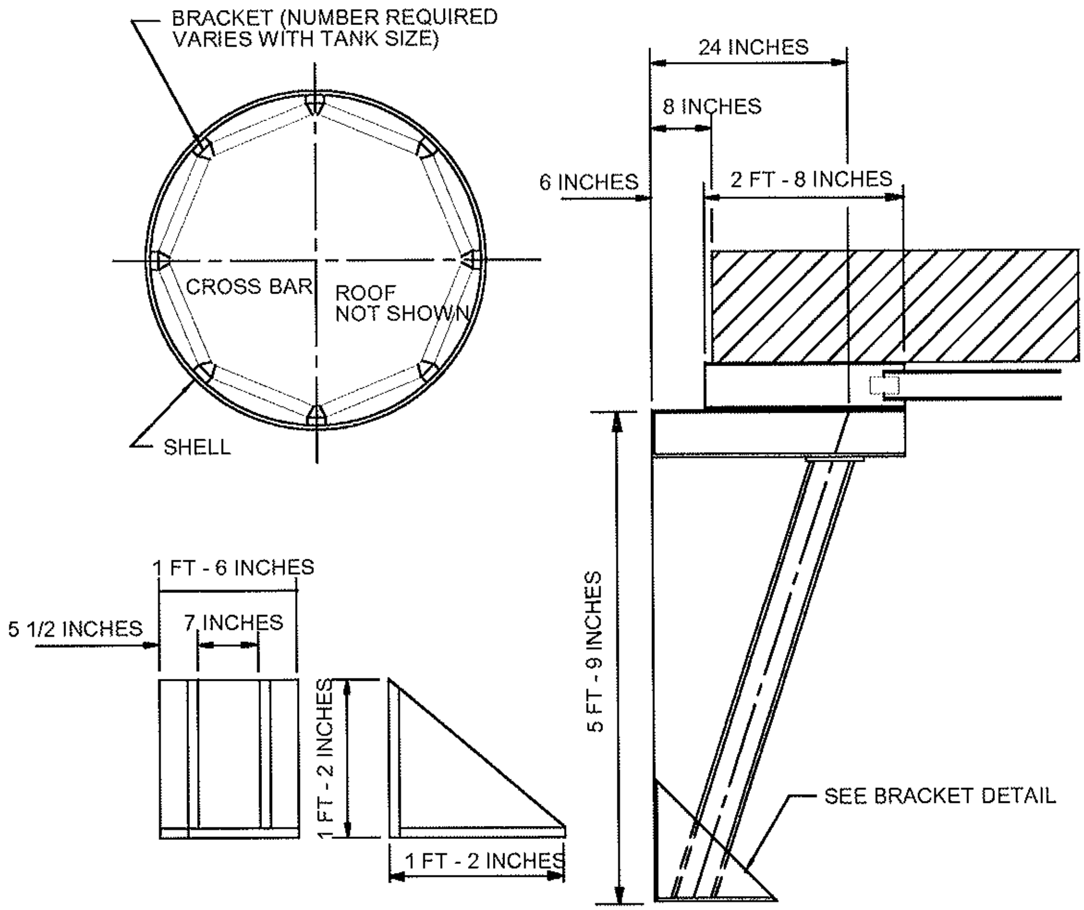

Figure C-10. Floating Roof Support

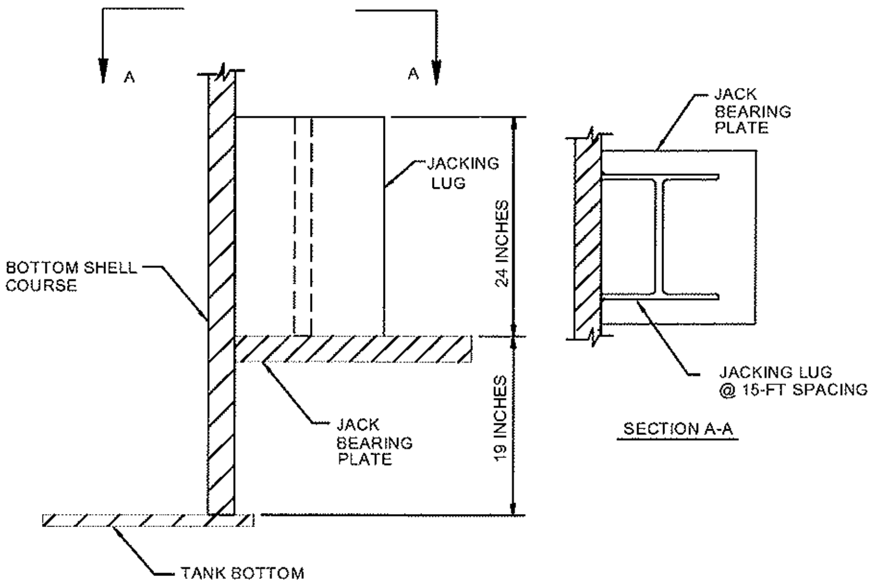

Figure C-11. Jacking Lugs Used on Large Tanks

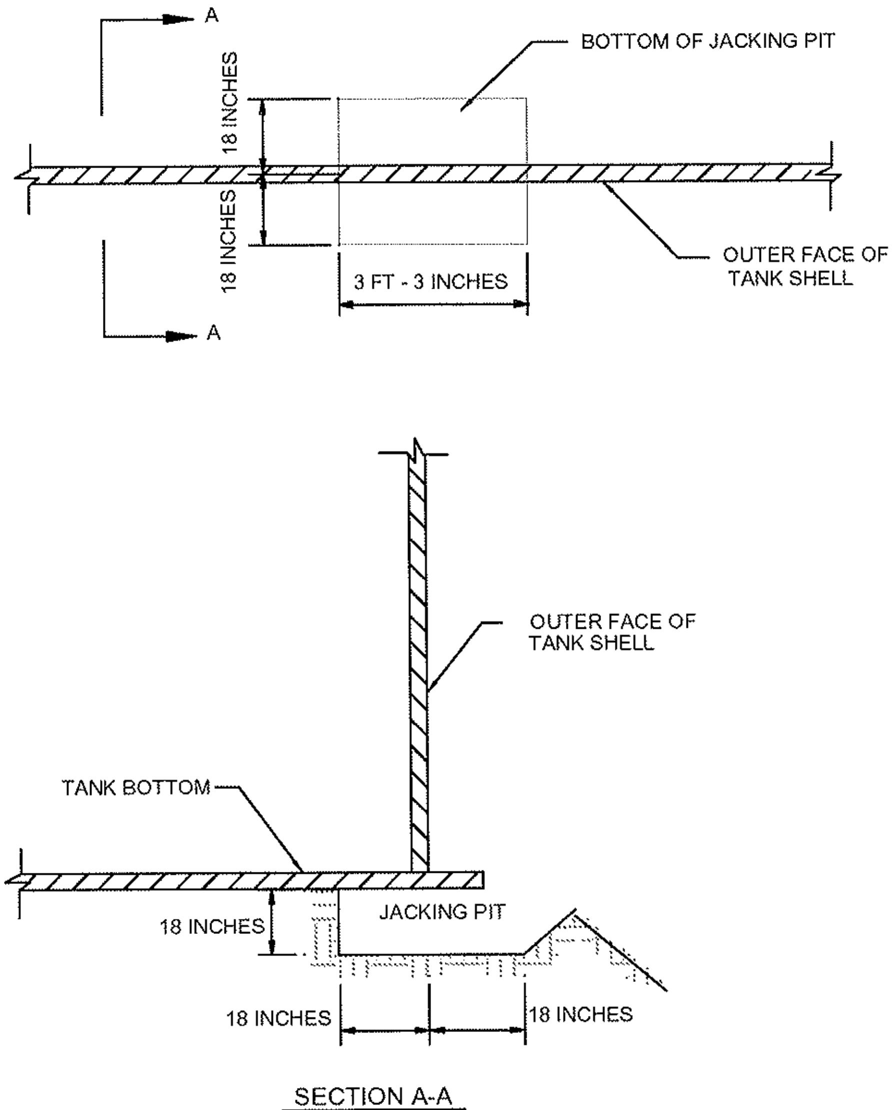

Figure C-12. Jacking Pit Dimensions