Best Practice

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Storage Tank Ringwall Foundation Design Guide

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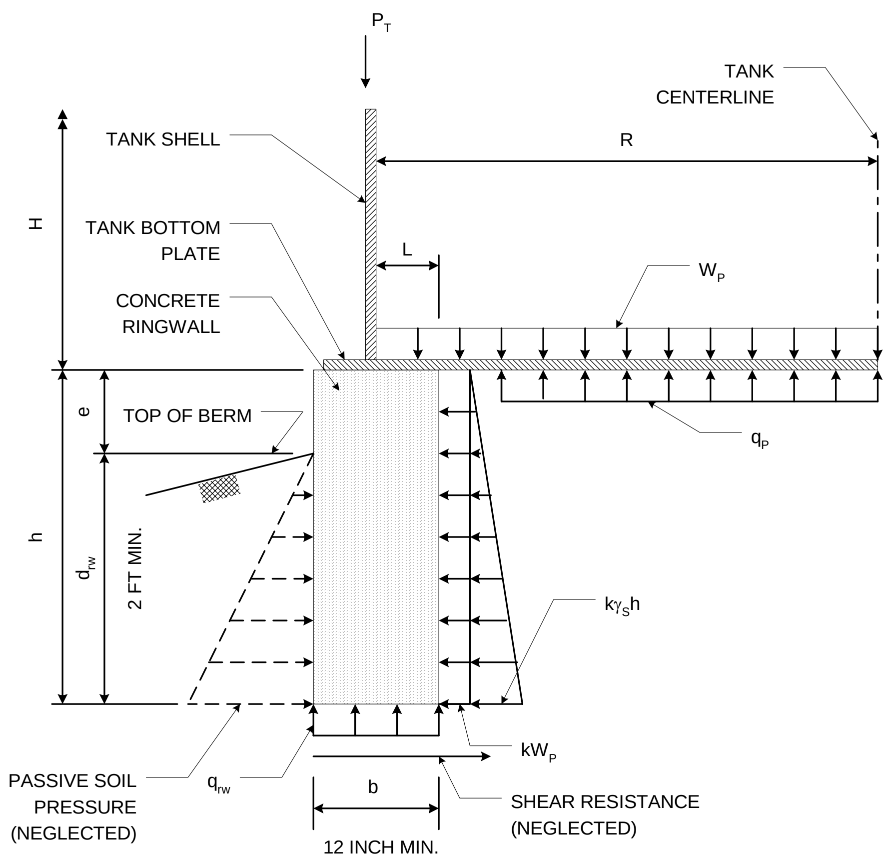

Developed by: Hisham Abu-Adas
Civil Engineering Unit/M&CED
Consulting Services Department

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Primary contact: Abu-Adas, Hisham A. on phone 966-3-8746908

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1 Introduction

1.1 Purpose

The purpose of this practice is to provide the engineer and designer with guidelines for the analysis and design of storage tank ringwall foundations for use by engineers working on Saudi Aramco projects and Saudi Aramco engineers.

1.2 Scope

This design guide defines the minimum requirements for the analysis and design of storage tank ringwall foundations normally encountered in process industry facilities at Saudi Aramco sites. It covers general design philosophy and requirements to be used in the analysis and design of ring wall foundations. Section 2.0 of this instruction includes reference codes and Saudi Aramco standards. General tank foundation information and recommendations are presented, relevant design formulas are derived. In cases where this guideline conflicts with these references, the conflict shall be immediately brought to the attention of the project engineer. Process Industry Practice *PIP* STE03020 *"Guidelines for Tank Foundation Designs"* forms the basis for the development of this design guide.

1.3 Disclaimer

The material in this Best Practices document provides the most correct and accurate design guidelines available to Saudi Aramco which complies with international industry practices. This material is being provided for the general guidance and benefit of the Designer. Use of the Best Practices in designing projects for Saudi Aramco, however, does not relieve the Designer from his responsibility to verify the accuracy of any information presented or from his contractual liability to provide safe and sound designs that conform to Mandatory Saudi Aramco Engineering Requirements. Use of the information or material contained herein is no guarantee that the resulting product will satisfy the applicable requirements of any project. Saudi Aramco assumes no responsibility or liability whatsoever for any reliance on the information presented herein or for designs prepared by Designers in accordance with the Best Practices. Use of the Best Practices by Designers is intended solely for, and shall be strictly limited to, Saudi Aramco projects.

1.4 Conflicts with Mandatory Standards

In the event of a conflict between this Best Practice and other Mandatory Saudi Aramco Engineering Requirement, the Mandatory Saudi Aramco Engineering Requirement shall govern.

2 References

This Best Practice is based on the latest edition of the references below, unless otherwise noted. Short titles will be used herein when appropriate.

2.1 Saudi Aramco References

Saudi Aramco Engineering Standards (SAES)

SAES-A-112	*Meteorological and Seismic Design Data*
SAES-A-113	*Geotechnical Engineering Requirements*
SAES-A-114	*Excavation and Backfill*
SAES-A-204	*Preparation of Structural Calculations*
SAES-M-001	*Structural Design Criteria for Non-Building Structures*
SAES-Q-001	*Criteria for Design and Construction of Concrete Structures*
SAES-Q-005	*Concrete Foundations*

Structural Design Best Practice

SABP-Q-001	*Anchor Bolt Design and Installation*

2.2 Industry Codes and Standards

American Concrete Institute (ACI)

ACI 318 -02	*Building Code Requirements for Reinforced Concrete and Commentary*

American Society of Civil Engineers (ASCE)

ASCE 7-02	*Minimum Design Loads for Buildings and Other Structures*

American Petroleum Institute (API)

API STD 650	*Welded Steel Tanks for Oil Storage (incl. Appendices B, E & I)*

Process Industry Practices (PIP)

PIP STE03020 *Guidelines for Tank Foundation Designs*

2.3 Other References

Stability of Steel Oil Storage Tanks by Duncan, J. M and D'Orazio, T.B.

Journal of Geotechnical Engineering, Vol. 110, No. 9, Sept. 1984

3 General

3.1 The design and specifications for construction of storage tank ringwall foundations shall be adequate for the structure intended use, in accordance with Appendix B of API STD 650 and commonly accepted engineering practice, Saudi Aramco Standard SAES-Q-005 (Section 4.0), and this guideline.

3.2 The types of storage tank normally encountered in industrial plants have cylindrical shells, essentially flat bottoms, and either cone roofs or floating roofs. The height of these tanks generally does not exceed 60 feet and the diameter generally ranges from 10 feet to 360 feet.

3.3 Concrete ringwalls are used to help distribute heavy shell loads such as from large dome roof tanks where the roof is supported entirely by shell. Concrete ringwalls are also used when the tank must be anchored for internal pressure, earthquake, or wind loads.

3.4 Ringwall foundation design shall conform to this engineering guideline and to applicable requirements of the soil report.

3.5 A geotechnical investigation is required for all new structures and foundations as described in SAES-A-113. (Ref. SAES-Q-005, Para. 4.1.1)

3.6 The allowable soil bearing pressure shall be based on the results of the geotechnical investigation, and a consideration of permissible total and differential settlements. Soil pressures shall be calculated under the action of vertical and lateral loads using load combinations that result in the maximum soil pressures. The maximum soil pressure shall not exceed the applicable allowable value. (Ref. SAES-Q-005, Para. 4.1.2)

3.7 Foundations shall be founded on either undisturbed soil or compacted fill. In the case of foundations supported on compacted fill, the geotechnical investigation and/or SAES-A-114 shall govern the type of fill material and degree of compaction required. (Ref. SAES-Q-005, Para. 4.1.3).

3.8 In areas of firm soils, the following site preparation steps are recommended:

a) Strip the site of all topsoil and organic material.

b) Set the top of ringwall a minimum of 6 inches above finished grade elevation to ensure adequate drainage. Coordinate this with piping group.

c) Slope interior tank pad surface to match tank bottom. A minimum of 1 inch to 10 feet is recommended (taking into account settlement and differential settlement between the tank center and the ring wall) to ensure adequate tank drainage.

d) Provide a concrete ringwall to reduce differential settlement along the circumference of the tank shell.

3.9 In areas of weak compressible soils, special foundations and design procedures or soil improvement may be required to prevent failures or excessive settlement. Please refer to project soil investigation recommendations.

3.10 Ringwall depth below grade should be determined from allowable soil bearing pressures.

3.11 For tanks utilizing interior roof supports, the tank bottom shall be reinforced as required to distribute column loads and reduce local settlement.

3.12 When the tank diameter is less than 20 feet, the engineer should consider the use of a solid concrete octagon as opposed to a ringwall foundation.

3.13 The design and construction of all concrete foundations shall comply with the requirements of SAES-Q-001. (Ref. SAES-Q-005, Para. 4.3.1)

3.14 The design concrete compressive strength of concrete shall be 27.6 MPa (***4000 psi***) at 28 days. (Ref. SAES-Q-005, Para. 4.3.2.b)

4 General Design Considerations

4.1 Load Types and Applications

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

4.1.1 Dead Load

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

4.1.2 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.

4.1.3 Vapor Space Design Pressure and Pneumatic Test Pressure

4.1.3.1 Internal pressure on the roof and surface area of the contents is identical; however, the bottom plate (typically ¼ 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.

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

4.1.3.3 Tanks with internal pressure generally require foundations with anchor bolts.

4.1.4 Snow Load

Normally, snow loads do not exist on typical Saudi Aramco facilities. However, for tanks in snow regions, the weight of snow should be considered in the foundation design. Snow load should be calculated in accordance with ASCE 7.

4.1.5 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.

4.1.6 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 shall be calculated in accordance with the requirements of SAES-A-112 and SAES-M-001 'Structural Design Criteria for Non-Building Structures" and ASCE 7 'Wind Load and Anchor Bolt Design for Buildings and Other Structures".

4.1.7 Earthquake Load

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

4.1.7.1 Earthquake forces should be calculated in accordance with ASCE 7 and *API STD 650*, Appendix E.

4.1.7.2 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.

4.1.7.3 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 STD 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.

4.1.7.4 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.

4.1.8 Shear Loads

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

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

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

4.1.8.4 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.

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

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

4.1.8.7 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.

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

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

4.1.9 Settlement

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

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

4.1.9.3 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 Ground Supported Storage Tank Load Combinations

Load combinations for ground supported storage tanks shall be taken from *API STD 650*. Load combinations from *API STD 650* and modified for use with *ASCE 7* loads and are shown in Table 9 below.

Table 9 – ASD Load Combinations for Ground Supported Storage Tanks

Load Combination	Description
1. $D_s + D_o + P_i$	Operating Weight + Internal Pressure[a]
2. $D_s + D_t + P_t$	Test Weight + Test Pressure
3. $D_s + (D_e \text{ or } D_o) + W + 0.4P_i^b$	Empty or Operating Weight + Wind + Internal Pressure[a]
4. $D_s + (D_e \text{ or } D_o) + W + 0.4P_e^b$	Empty or Operating Weight + Wind and External Pressure
5. $D_s + D_o + L + 0.4P_e^b$	Operating Weight + Live + External Pressure
6. $D_s + (D_e \text{ or } D_o) + 0.4\,L + P_e$	Empty or Operating Weight + Live + External Pressure
7. $D_s + D_o + E_o^c + 0.4\,P_i^b$	Operating Weight + Earthquake Internal Pressure (Earthquake uplift case)
8. $D_s + D_o + E_o^c$	Operating Weight + Earthquake

D_s, D_e, D_o, and D_t, where:

D_s = Structure dead load is the weight of materials forming the structure foundation, soil above the foundation resisting uplift. D_s does not include the empty weight of the tank which is covered below.

D_e = Empty dead load - Empty weight of tanks and piping. For checking uplift and components controlled by minimum loading, the corroded metal weight (when a corrosion allowance is specified) should be considered as the empty dead load.

D_o = Operating dead load is the empty weight of tanks, piping, and maximum weight of contents (fluid load) during normal operation. Operating dead load for a ground supported storage tank is made up of the metal load from the tank shell and roof vertically applied through the wall of the tank in addition to the fluid load from the stored product. The fluid load acts through the bottom of the tank and does not act vertically through the wall of the tank. Therefore, the metal dead load and the fluid load must be used separately in design.

D_t = Test dead load is the empty weight of tanks, and/or piping plus the weight of the test medium contained in the system.

P_i, P_e, and P_t, where

P_i = Design Internal Pressure

P_e = External Pressure

P_t = Test Internal Pressure

L = Live load

W = Wind load

Eo = Earthquake load considering the operating load case

Notes:

a. For internal pressures sufficient to lift the tank shell according to the rules of *API STD 650*, tank, anchor bolts, and foundation shall be designed to the additional requirements of *API STD 650* Appendix F.7.

b. If the ratio of operating pressure to design pressure exceeds 0.4, the owner should consider specifying a higher factor on design pressure in load combinations 3, 4, 5, and 7 above.

c. Earthquake loads for *API STD 650* tanks taken from *ASCE 7* "bridging equations" or from *API STD 650* already include the 0.7 ASD seismic load factor.

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 should be used.

4.3.1.2 For mall tanks (20-ft in diameter or less), concrete slab 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.

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 4,000 psi at 28 days.

5.1.6 Concrete and reinforcement should be specified in accordance with *ACI 318* and *API STD 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 ½ 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 Concrete Slab Foundation

5.2.1 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.

5.2.2 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.

5.2.3 Structural concrete should be designed in accordance with SAES-Q-001.

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

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

5.2.6 The concrete slab should be heavy enough to resist overturning forces with a safety factor in accordance with SAES-Q-005.

5.3 Ringwall Foundation Design Procedures

5.3.1 General

5.3.1.1 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 STD 650* and *ACI 318* that should be used to design a ringwall foundation for a tank.

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

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

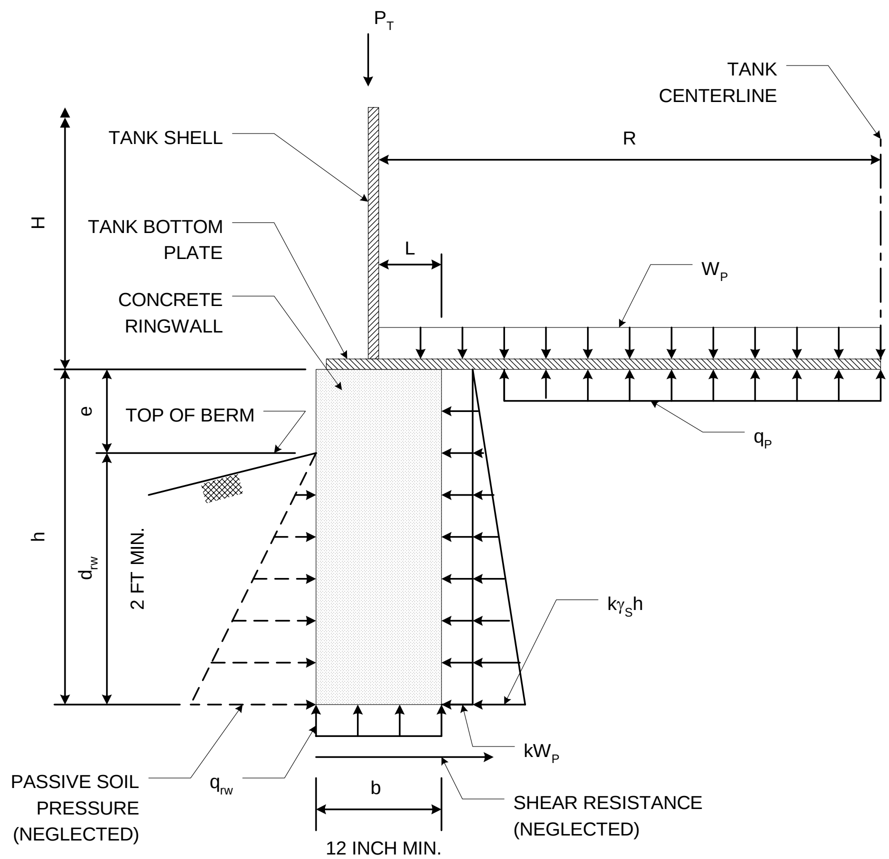

Figure 3 – Ringwall Loading for Unanchored Tank

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
P_T = 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 from Soil Investigation Report, 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

5.3.2 Foundation Sizing

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

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

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

$$b = \frac{P_T + W_P(L)}{q^a + (h-e)\gamma_S - h\gamma_C} \qquad \textbf{(Eq. 1)}$$

5.3.2.4 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 ≤ 80	4
80 < D < 150	5
D ≥ 150	6

5.3.2.5 The minimum values of L shown in Table 2 are 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.

5.3.2.6 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.

5.3.2.7 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.

5.3.3 Hoop Tension

5.3.3.1 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.

5.3.3.2 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.

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

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

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

$$A_S = \frac{1.6T_h}{0.9\, f_y} \qquad \textbf{(Eq. 3)}$$

5.3.4 Twist

5.3.4.1 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.

5.3.5.2 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.

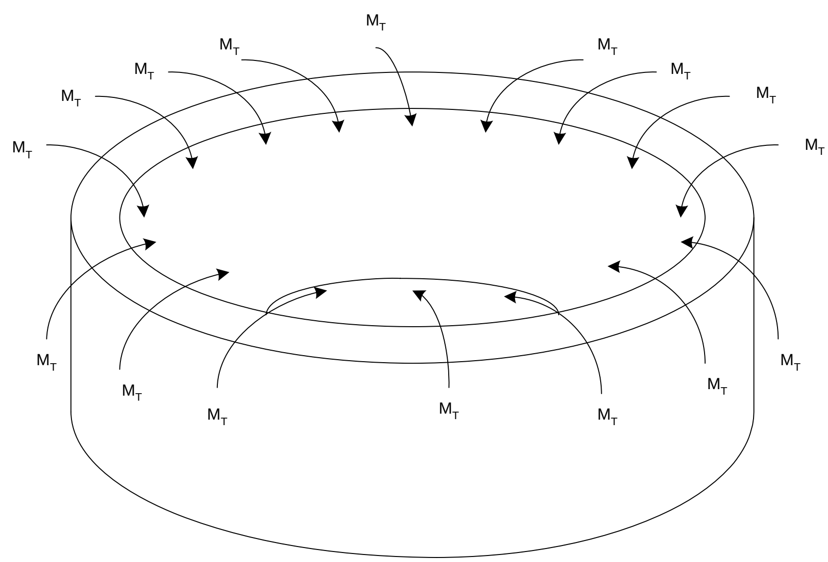

Figure 4 – Uniform Twist Moment

5.3.4.3 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.

5.3.4.4 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.

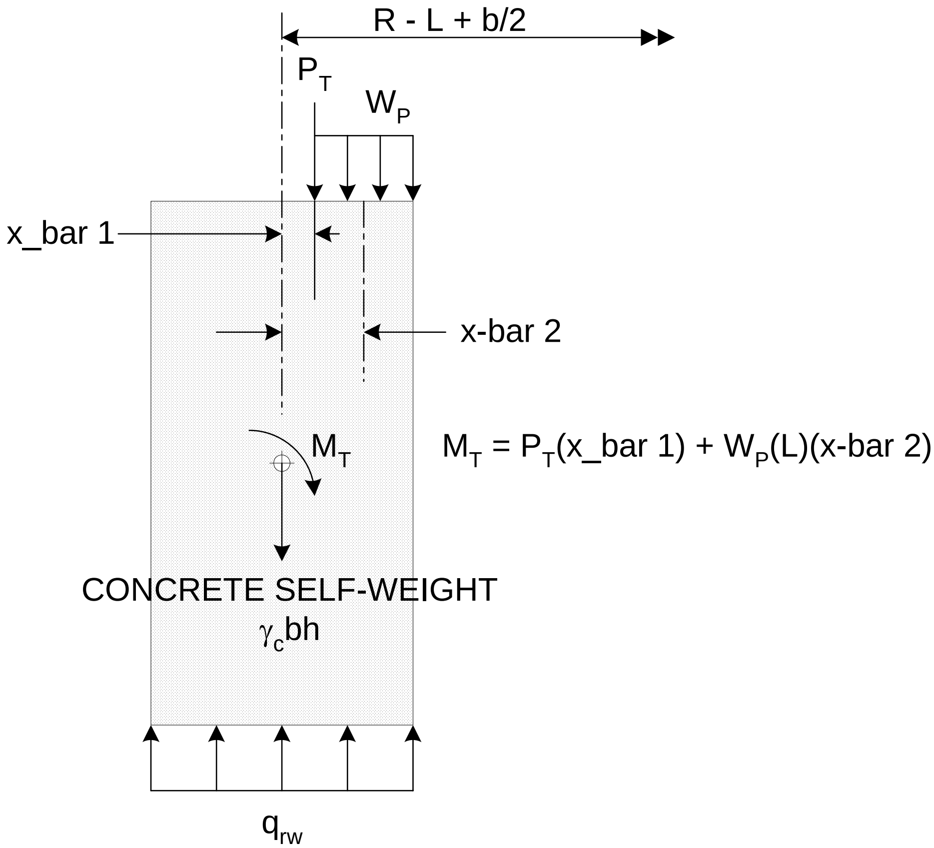

Figure 5 – Calculation of Twist Moment

5.3.4.5 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.

4.3.5.6 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*:

$$M = M_T \left(R - L + \frac{b}{2} \right) \qquad \textbf{(Eq. 4)}$$

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

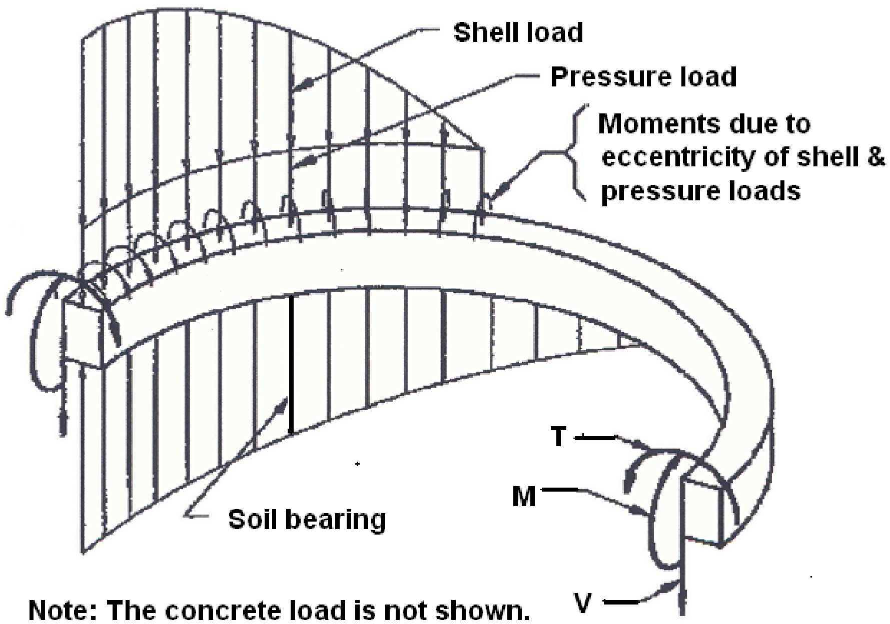

Figure 6 – Load Distribution under Seismic or Wind

5.3.4.8 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.

5.3.4.9 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.

5.3.4.10 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.

5.3.5 Base Shear and Sliding

5.3.5.1 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.

5.3.5.2 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.

5.3.5.3 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.

5.3.6 Minimum Reinforcing Steel Requirements

5.3.6.1 In accordance with *API STD 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

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

5.5.3.3 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.

5.3.7 Special Ringwall Foundation Considerations for Seismic Loads

5.3.7.1 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.

5.3.7.2 Tank ringwall foundations are normally designed for the ringwall moment in accordance with *API STD 650*. 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 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.

5.3.7.3 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 STD 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 STD 650* Tolerances

To achieve the *API STD 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 ±¼ 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.

6.3 Anchoring

6.3.1 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 STD 650*, Appendix E.

6.3.2 If it is not possible to keep tank's H/D ratio low enough, anchors can be required.

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

6.4 Special Considerations in Weak Soils

In areas of weak compressible soils, special foundations and design procedures or soil improvement may be required to prevent excessive settlements. As a general guideline, if the differential settlement approaches ½ inch to ¾ inch in a distance of 30' – 40' of the shell circumference of an ordinary flat bottom tank, a detailed evaluation of the soil pressure shall be made. Under no circumstances shall the soil pressure exceed the allowable.

Consideration shall also be given to the location of drain out nozzles at the bottom of the tank and appropriate block out details provided in the concrete ring wall foundation. This will prevent any interference with the drain out nozzle flanges.

7 Anchor Bolts

Anchor bolts design requirements shall be in accordance with Para. 4.7 of SAES-Q-005 and SABP-Q-001.

Revision Summary

31 August, 2002	New Saudi Aramco Best Design Practice SABP-Q-005.
26 March 2008	Major revision.(Revised to reflect updated Saudi Aramco Standards and Industry Codes & Practices.)

Appendix A – Ringwall Foundation Design Examples

Example 1 – (In Seismic Zone)

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 STD 650*

Foundation Data:

fy = 60 ksi (rebar)

f'c = 4 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

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

$$b = \frac{P_T + W_P(L)}{q^a + (h-e)\gamma_S - h\gamma_C} \qquad \textbf{(Eq. 1)}$$

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

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

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

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

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 *Table* 9:

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

By inspection, Load Combination 2 will control.

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

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

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

$$A_S = \frac{1.6 T_h}{0.9\, f_y} \qquad \textbf{(Eq. 3)}$$

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

Determine Twist Steel

Determine factored twist moment using 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

$$M = M_T\left(R - L + \frac{b}{2}\right) \qquad \textbf{(Eq. 4)}$$

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

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

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

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

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

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

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

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

$$\rho = \frac{0.85(4000)}{60000}\left(1 - \sqrt{1 - \frac{2(160.0)}{0.85(4000)}}\right) = 0.00273$$

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

$$\rho_{\max} = \frac{4}{3}(0.00273) = 0.00364$$

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

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

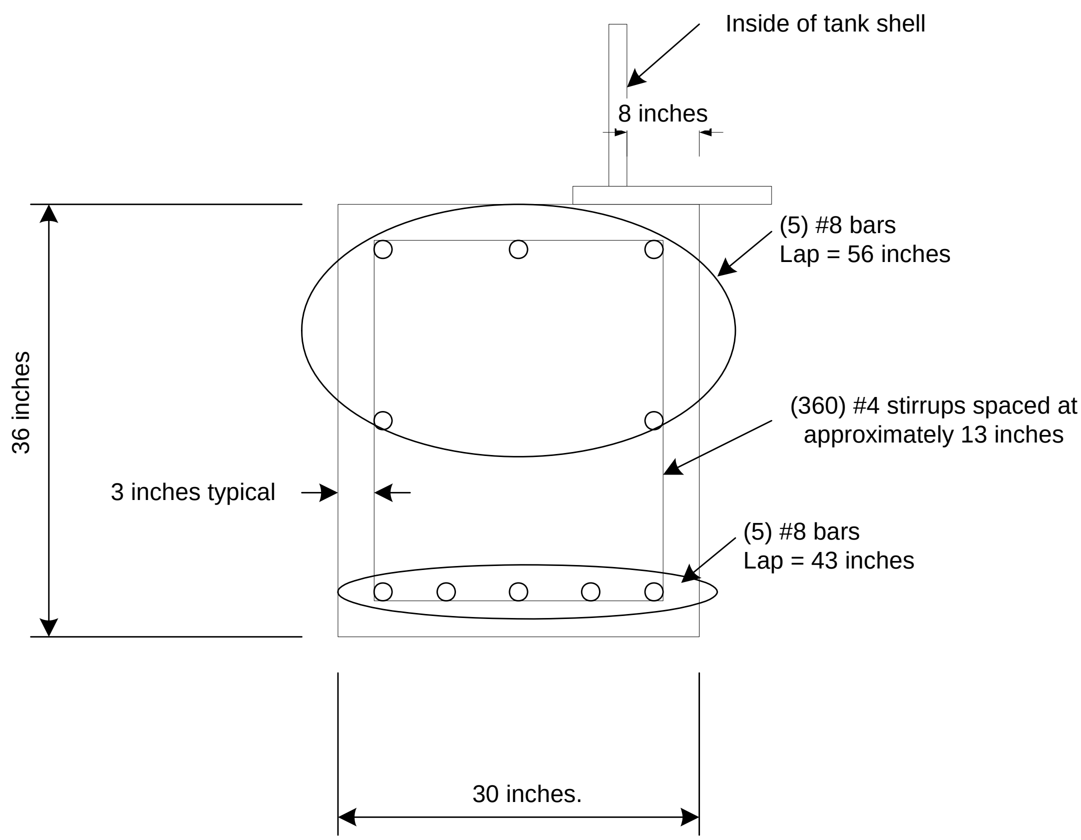

Figure A-1 – Final Design - Example 1

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 – No Seismic Load

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 STD 650*

Snow: none

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

Foundation Data:

fy = 60 ksi (rebar)

f’c = 4 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 Table 9:

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

$$b = \frac{P_T + W_P(L)}{q^a + (h-e)\gamma_S - h\gamma_C} \qquad \textbf{(Eq. 1)}$$

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

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

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

∴ **use b = 1 ft – 6 inches.**

Determine Hoop Steel

Determine applicable load combinations (LC) from Table 9:

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

By inspection, Load Combination 2 will control.

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

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

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

$$A_S = \frac{1.6 T_h}{0.9\, f_y} \qquad \textbf{(Eq. 3)}$$

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

Determine Twist Steel

Determine factored twist moment using 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

$$M = M_T\left(R - L + \frac{b}{2}\right) \qquad \textbf{(Eq. 4)}$$

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

b = 18 inches d = 60 – 3 – 1.128 / 2 = 56.4 inches

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

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

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

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

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

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

$$\rho = \frac{0.85(4000)}{60000}\left(1 - \sqrt{1 - \frac{2(12.17)}{0.85(4000)}}\right) = 0.000203$$

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

$$\rho_{\max} = \frac{4}{3}(0.0000203) = 0.000271 \quad \text{governs}$$

$$A_{S\ TWIST} = 0.000271(18)(57) = 0.28 \text{ inch}^2$$

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

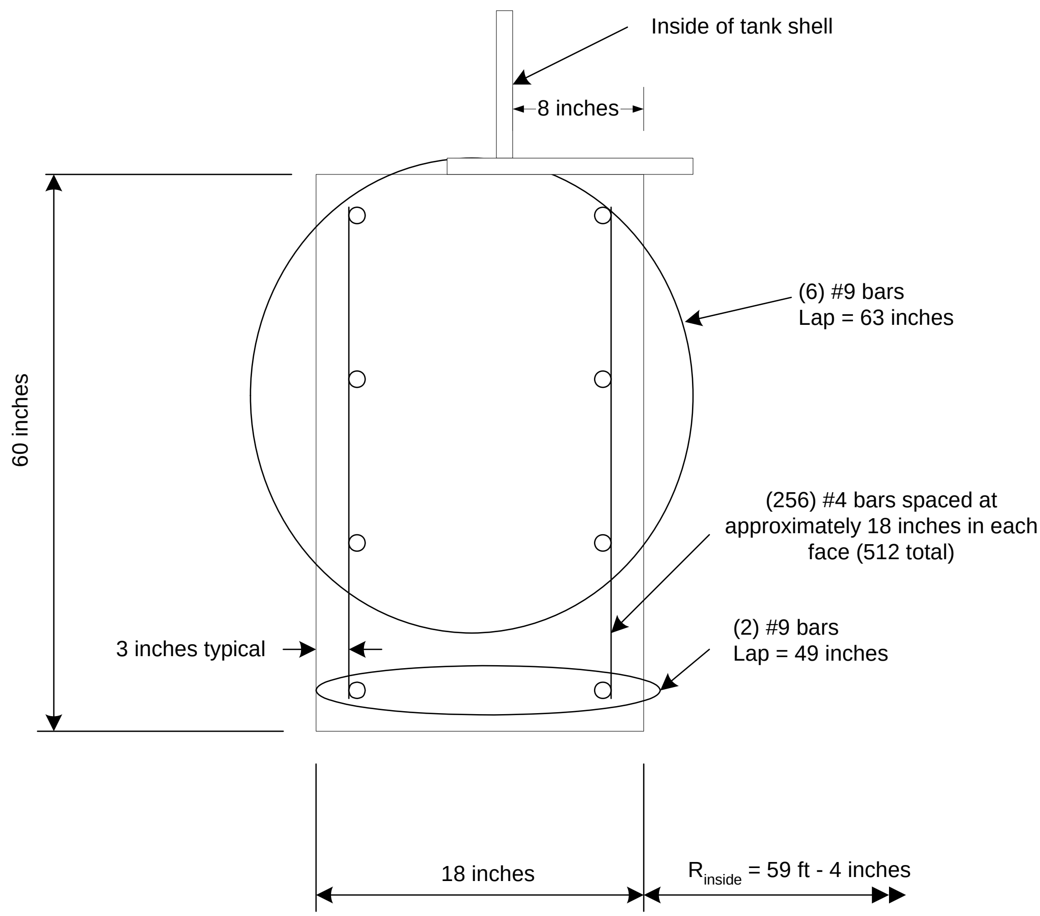

Figure A-2 – Final Design - Example 2

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*.