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a small, thick-walled tank on a shallow foundation loaded to the point of tipping will generally cause a bearing capacity failure in the soil in the course of tipping over. Ensuring that a bearing capacity failure does not occur under the design loads will also ensure that the tank does not tip over.

During the OBE and SSE events, overturning resistance is provided by the self-weight of the outer and inner tank. The excess of the roof's weight over the pressure of the gas supporting it, if any, will also resist uplift, but the gas pressure can also contribute to wall uplift if the gas pressure exceeds the weight of the roof. Non-uniform hydrodynamic base pressures should be considered in determining the moment-couple on the foundation. The weight of the foundation can be included in overturning resistance if the tank is adequately anchored to the foundation.

Overturning resistance will generally exceed overturning moments in the tanks treated by this code because of the tanks' weights, large diameters, and proportions. Excessive overturning moment will generally cause a bearing capacity failure of the soil, overstress in the foundation slab, or severe wall deformation first before overturning of the tank could occur. Where overturning is a possible failure mode, the factor of safety against overturning should be not less than shown in Tables 10.3.2.4 and 10.4.4.

10.3.3.1 Calculations shall be performed to determine the effects of overturning moments on the tank both when full and empty, and resistance to the effects shall be provided.

10.3.3.2 The combined effect of overturning moment and the tendency for gas pressure against the roof to lift the walls shall be considered in determining the need for uplift resistance.

R10.3.3.2 Civil engineering structures normally have a structural element in direct contact with ground (for example, foundation, piles, and mats). When the external forces, such as earthquakes, act on these systems, neither the structural displacements nor the ground displacements are independent of each other.

Internal gas pressure in the tank does not have any effect on total vertical load, global overturning moment, or global overturning resistance, but it affects the distribution of foundation pressure as well as stress distribution in the tank walls and roof under all loading conditions. Therefore, the contribution of gas pressure to uplift of the wall should be considered in combination with any uplifting effect from overturning moment.

10.3.3.3 Shallow foundations shall be sized to resist uplift forces where needed.

10.3.3.4 Anchorage details shall be capable of accommodating movement of the tank wall caused by thermal changes.

10.3.4 *Sliding resistance*—The minimum factor of safety against sliding shall not be less than 1.5 for wind and OBE loading cases, and 1.2 for SSE loading cases.

R10.3.4 Sliding resistance may be provided by frictional forces due to self-weight of the combined tank system. Sliding resistance should be checked over a range of conditions, including tank empty and tank full.

For granular soil, sliding resistance is governed by the coefficient of friction between the soil and the foundation

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plus the roughness and shape of the foundation. For cohesive soil, sliding resistance is governed by the adhesion of the soil to the foundation and the shear strength of the soil plus the shape of the foundation.

The calculated resistance to sliding of a tank on a shallow foundation may be taken as a coefficient of friction times the weight of tank and contents including the reduction in normal force due to vertical earthquake. The maximum ultimate coefficient of friction should be tan30 degrees unless testing validates a higher value. The maximum allowable frictional resistance under wind lateral load is equal to 0.40 with a safety factor of 1.5. The coefficient of friction should consider the materials underlying the tank bottom. Alternatively, shear keys may be constructed within the foundation to mobilize passive pressure effects and increase the sliding capacity of the foundation.

10.3.5 *Settlement*—The effect of immediate and long-term settlement on strength and serviceability shall be considered in the design of foundation base slabs, containments and connections to adjacent equipment, plant piping, and other systems.

R10.3.5 The settlement requirements in this code serve as guidance and should be agreed upon with the tank supplier and the owner. Settlement calculations should assume that the tank is full for its design life when considering the effect of settlement in the design of primary- and secondary-containment tanks. The range of possible settlement needs to be considered when designing connections to adjacent equipment, plant piping, and other systems that connect to containments.

10.3.5.1 The maximum settlements during the life of the tank shall be within the permissible settlement limits for the tank and associated tank components.

R10.3.5.1 Requirements for limiting settlement will generally govern the foundation design of large refrigerated tanks with foam-glass insulation in the bottom slab. Predictions of settlement using the tank's weight, the compressibility of the soil, and assumed stress distributions in the soil may limit allowable bearing pressure to values lower than those calculated with the factors listed in Table 10.3.2.4. Such a limitation may be necessary for an overconsolidated clay simply because of the long-term static load of a full tank and the soil's consolidation. The effects of cyclic loading should be considered for a large shallow foundation on sand. Settlements can accumulate in loose or medium-dense sand with the repeated emptying and filling of the tank even though the pressures during service are well below the ultimate bearing pressure.

10.3.5.2 Unless otherwise specified in the project documents, maximum limits for settlements of concrete shallow foundations shall be as follows:

a) Uniform settlement shall be permitted, provided the other provisions of the section are met and the connecting piping system will accommodate the settlement.

b) Differential settlement or uniform (planar) tilting (when the tank foundation tilts uniformly to one side) shall be limited to a maximum of 1/500.

c) Dishing settlement measured along a radial line from the outer perimeter to the tank center shall be limited to a maximum of 1/300.

R10.3.5.2 Restricting the dishing settlements to 1/300 maintains the bending curvatures within acceptable limits so that insulation materials (foam-glass) are not damaged.

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d) Foundation settlement around the perimeter of the tank shall be limited to the lesser of: 1) 1/500; and 2) the maximum settlement limit calculated for the uniform tilting of the tank.

10.3.5.3 Computation of settlements shall take into consideration the effects of adjacent tanks, tank foundation/wall stiffness, fill surcharge, soil stiffness, time required for consolidation, soil variability, and the reliability of the site investigation.

10.4—Design requirements for deep foundations

10.4.1 *General requirements*

10.4.1.1 The selection and design of the deep foundation system shall be conducted by the foundation designer in close cooperation with the geotechnical engineer and the structural engineer.

10.4.1.2 The selection and design of the deep foundation system shall be based on a geotechnical investigation of the in-place foundation conditions, and shall take into account the engineering properties of those foundations.

10.4.2 *Piles*

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R10.3.5.3 Fill surcharge is the weight of fill material over the original grade. It does not include preload surcharge for ground modification.

R10.4—Design requirements for deep foundations

R10.4.1.1 The structural engineer should use the geotechnical information combined with engineering properties of the deep foundation systems to design and specify the deep foundation system. The structural engineer, in cooperation with the geotechnical engineer, should specify such performance testing and testing frequency to ensure that the constructed foundation is adequate to perform the functions required.

Considerations in the design of the deep foundation system besides the soil engineering properties should include material availability, potential contractor capability, constructibility, equipment availability, and local requirements. The performance-testing program should be designed by the structural engineer and the geotechnical engineer to establish the most cost-effective pile design.

R10.4.1.2 Because deep foundations are typically constructed of numerous individual components whose capacities are additive to provide the strength of the foundation, great care in geotechnical information gathering, design, construction, and performance testing of all components or representative samples is required.

R10.4.2 Piles can include both driven piles and cast-in-place concrete piles. An early pile selection and testing program to test and determine pile load characteristics, pile installation methods, and procedures can be beneficial. Such a program is most effectively conducted shortly after the geotechnical investigation. Where possible, all piles installed under the program should be electronically monitored and evaluated. Tested piles may be incorporated into the final design. The program, however, does not represent the start of construction as it is an extension of testing.

A static analysis should be performed using an acceptable and proven method for the area where the piles are being driven. Effects such as additional fill, water table level, pile group efficiency, corrosion protection, and pile splicing should be taken into consideration when the pile type and length are chosen.

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10.4.2.1 *Driven piles*—Dimensional tolerances for the production of precast concrete piles, both solid and hollow, shall be in accordance with PCI MNL-116-99.

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R10.4.2.1 Driven piles can include open or closed steel pipe piles, H-piles, single or spliced solid prestressed concrete piles, and concrete cylinder piles. Concrete piles with cast-in-place splicing devices may reduce transportation and handling requirements significantly enough to justify the general use of splices for long piles.

Pile blow counts for driven piles should be recorded electronically. A pile inspector, qualified as per project specifications, should be present during fabrication and driving of all piles. Examples of adequate qualifications can include completion of a U.S. Federal Highway Administration's (FHWA's) pile inspector course together with experience in inspecting piles acquired by working under previously qualified pile inspectors.

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10.4.2.2 *Cast-in-place piles*

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R10.4.2.2 Cast-in-place piles include drilled caissons, drilled piers, auger-cast-in-place piles, and auger-displacement-pressure-grouted piles (ADPGP). Proprietary methods of construction are often used.

Cast-in-place pile safety factors are usually higher than those for driven piles due to higher uncertainty in the constructed condition. Additional guidance may be found in ASCE 20-96, ACI 336.1R, PIP STS02380, and PIP STE02465

Construction procedures for cast-in-place piles should be developed in advance with the piling sub-contractor's advice to address construction issues such as;

a) Dense reinforcement that is difficult to install in augered-cast-in-place piles, or difficulty in consolidating concrete in drilled piers around dense reinforcement
b) Grout for augered-cast-in-place piles with sufficient performance to allow adequate time for placement of grout and reinforcement cage
c) Procedures for field bending of reinforcement, if required
d) Connections to the structure
e) Inspection methods and placement of instrumentation such as inspection tubes

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10.4.2.3 *Test pile program*—Number and type of tests to be conducted shall be specified by the geotechnical and structural engineer.

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R10.4.2.3 Economics and the requirement for safety in refrigerated liquefied gas (RLG) tank design will typically justify a comprehensive test pile program to validate the static analysis. The program should include a pile driving simulation to develop the driving criteria, dynamic monitoring to adjust the driving criteria, and an ASTM or similar static load test to validate or finalize the pile design. Depending on the number of piles required, it may be economically justified to perform a pile driving simulation and dynamically monitor installation of selected piles to verify hammer performance and adjust driving criteria. Safety factors and the number of piles tested and monitored may be adjusted based on a reliability analysis that considers the uncertainty in loads and the variability of soil conditions.

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10.4.2.4 *Pile driving effects*

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R10.4.2.4 For large pile groups of closed pipe piles or solid prestressed concrete piles, predrilling may be considered to reduce the driving effort and to reduce heave. The

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use of open-ended pipe piles will also reduce the heave and lateral movement of an installed pile due to installation of an adjacent one. A driving pattern that moves outward from the center of the pile group should be considered to limit the effect on other piles.

10.4.3 *Ultimate strength of single piles*

10.4.3.1 Ultimate strength of single piles shall be based on the results of a geotechnical investigation and one of the following:

a) Application of generally accepted geotechnical and civil engineering principles to determine ultimate strength of the tip in end bearing, and side friction or adhesion
b) Static load testing in accordance with ASTM D1143
c) Other in-place load tests that measure end bearing and side resistance either separately or together
d) Dynamic testing in accordance with ASTM D4945

10.4.3.2 Where needed, ultimate axial tensile strength of single piles shall be determined by a test performed in accordance with ASTM D3689.

10.4.4 *Allowable pile capacity*

10.4.4.1 Allowable pile service load Q_a shall be the smaller value determined from:

a) Structural strength of the pile
b) Ultimate strength of single piles, Q_r, divided by minimum factors of safety from Table 10.4.4
c) Permissible total and differential settlement limits

R10.4.4.1 The safety factors in Table 10.4.4 are designed for use with nominal (unfactored) loads, and are intended to account for both the uncertainties in load and resistance in one factor. To avoid the overly conservative practice of simultaneously applying the maximum values of all dead loads, live loads, and environmental loads, the engineer should refer to Table 5.2.2 for load combinations, and Tables 7.2 and 7.3 for load factors.

Minimum safety factors in Table 10.4.4 may be reduced when: 1) justified by the geotechnical investigation and subsequent analysis; and 2) approved by the owner and engineer.

Table 10.4.4—Minimum factors of safety for deep foundations

Ultimate pile strength (as per code section)	Normal operation	Hydrostatic loading	Wind or OBE seismic	SSE seismic
10.4.3.1(a)	3.0	2.4	2.25	1.50
10.4.3.1 (b)	2.0	1.60	1.50	1.10
10.4.3.1 (c)	2.0	1.60	1.50	1.10
10.4.3.1 (d)	2.25	1.80	1.70	1.20

10.4.4.2 Allowable pile service load Q_a shall be reduced for group effects, down-drag, and other effects that reduce the strength of piling.

10.4.4.3 Quality control and construction inspection procedures for piles shall be developed before construction and agreed upon by the structural engineer, geotechnical engineer, contractor, and piling subcontractor.

10.4.5 *Overturning effects and uplift*

10.4.5.1 Analyses shall account for the redistribution of pile loads that is caused by lateral loading on the tank.

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10.4.5.2 The tendency for gas pressure against the roof to lift the walls shall be considered in determining the need for uplift resistance.

10.4.5.3 Factors of safety for the axial loads on piles under conditions of lateral loading on the tank and conditions of gas-induced uplift shall be in accordance with Table 10.4.4.

10.4.6 *Lateral load resistance*

10.4.6.1 The geotechnical engineer shall provide the necessary soil and pile parameters for analysis.

10.4.6.2 Sliding resistance of unanchored tanks supported by deep foundations shall comply with minimum factors of safety found in 10.3.4.

10.4.6.3 Allowable lateral load strength of a pile shall be determined by comparing predicted tank foundation deflections with allowable pile deflections and by limiting the bending stresses in piles.

10.4.6.4 Tank foundation deflections shall be predicted with a structural analysis that includes tank geometry and stiffness and the soil-structure interaction of the laterally loaded pile foundations.

10.4.7 *Settlement*

10.4.7.1 Unless otherwise specified in the project documents, maximum limits for settlements of deep foundations shall comply with 10.3.5.

10.4.7.2 An estimate of total and differential settlement of single piles and pile groups shall be determined by the geotechnical engineer.

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R10.4.5.2 Tension (uplift) resistance of piles structurally connected to the pile cap may be considered in combination with the effects of lateral loads.

R10.4.5.3 Safety factors for piles under tension loading are applicable to soil resistance of piles in compression and tension. Allowable stress levels for piles under load are given by ASCE/SEI 7. The ultimate strength for concrete piles under load is determined in accordance with ACI 350 or ACI 318.

R10.4.6 The effect of lateral loads on pile foundations should be evaluated. The geotechnical engineer should design the pile, pile group, or both, based on a determination of the lateral deflection of the pile head and distribution of resulting moment and shear along the pile shaft using a method of analysis that takes into account pile-soil elastic interaction, load duration, load repetition, structural restraint at the pile head, and the effect of group action.

Piles are categorized by whether they are short or long. In a short pile, the lateral strength is developed by rotation and the passive resistance of the surrounding soil. In a long pile, the lateral capacity is governed by the moments and stresses in the pile.

Lateral load analysis of piles is typically performed with computer programs using soil input parameters furnished by the geotechnical engineer. Alternatively, this task can be performed by the geotechnical engineer in preparing the report.

R10.4.6.1 In regions having seismic risk, the need to perform a lateral pile load test in accordance with ASTM D3966 should be evaluated by the geotechnical engineer.

R10.4.7.1 Deep foundations often cause the tank loads to have influence to a greater depth in the soil than would occur under a shallow foundation. Thus, the compressibility of soil layers at greater depth in the soil section will influence total long-term settlement, and should be addressed.

R10.4.7.2 The effect of fill on soils where deep foundations are used will cause the potential for additional load on the piles due to negative skin friction. In some instances, the

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load due to negative skin friction may necessitate lengthening of the piles, increasing the number of piles, or both.

Down-drag effects should be considered in predicting pile settlement. Drag load should be included in determining the required structural strength of the pile. Simple addition of the drag load to the dead load to get total axial load on the pile will often result in overly conservative designs, and is not recommended. For further guidance, refer to Fellenius (2006) and Briaud and Tucker (1997).

10.5—Ground improvement

Where required, ground improvement methods, materials, and procedures shall be developed by the geotechnical engineer in close cooperation with the structural engineer to increase bearing capacity to support the tank, reduce settlement to within the criteria of this standard, or improve seismic performance of the soils.

R10.5—Ground improvement

Soil improvement consists of densifying loose soils or strengthening loose or soft soils. Soil improvement will often be considered for use beneath a tank in two general cases. One case is where the native soils are too loose or soft for a shallow foundation to be adequate, but where the shallow foundation might be adequate if the soils are improved and where the combination is economically competitive with deep foundations. Another case is where deep foundations will be needed, but earthquake-related performance of the soil-foundation-tank system is inadequate without soil improvement.

The following methods should be considered to improve the soil to an acceptable performance level under these circumstances:

a) Removal of weak material and replacement with suitable fill material
b) Preloading with overburden to induce settlements
c) Preloading with overburden combined with improved subsoil drainage, such as wick drains or earthquake drains
d) Soil improvement through vibrocompaction
e) Deep soil mixing
f) Grout injection
g) Other methods as designed by the geotechnical engineer

10.6—Foundation details

10.6.1 *Groundwater*

10.6.1.1 The bottom of the tank shall be above the groundwater table or otherwise protected from contact with groundwater at all times.

10.6.1.2 Electrical heating conduits and other or exposed metal components of the outer tank bottom material in contact with soil shall meet at least one of the following requirements:

a) Selected to minimize corrosion
b) Coated, galvanized, or otherwise protected to minimize corrosion
c) Provided with a minimum of 3 in. of concrete cover
d) Where necessary, protected via cathodic protection system

10.6.1.3 Concrete parts of the outer tank bottom in contact with the soil shall meet the following requirements:

R10.6—Foundation details

R10.6.1.2 Cathodic protection does not have to be isolated, but the design should account for all metals and be electrically bonded to the system.

R10.6.1.3 ACI 222R, Section 4.4.3, notes that the results obtained from the ASTM C1202 test for chloride ion permeability are not always precise and, if concerns exist, should

a) Constructed of a concrete mixture with a rapid chloride permeability rating of less than 1000 coulombs charge passed as per ASTM C1202
b) Constructed of a durable concrete mixture as described in 6.6.5.9

be accompanied by a petrographic examination of the concrete. ACI 201.2R describes appropriate methods for obtaining durable concrete.

10.6.1.4 The area surrounding the tank shall be graded to drain away from the tank.

R10.6.1.4 Drainage away from the tank is important so that surface water and liquefied gasses flow to a drainage sump. This drainage will also aid in preventing a pool fire adjacent to the tank. Though the tank may be in a containment (bund) berm area, it is common practice to have the top of the tank pad at an elevation above the bottom of the containment area.

Consider that the design rainfall events may fill the containment berm (bund) area to an elevation above the level of the bottom of the tank until the water is drained. The heat in the water is likely to prevent damaging icing. The design of the tank foundation and tank appurtenances should consider exposure to water from flooding when appropriate.

10.6.1.5 Water or spilled refrigerated liquid shall not be allowed to pond adjacent to the tank.

10.6.1.6 The foundation shall bear at a depth below the shrinking-and-swelling zone or freezing-and-thawing zone, or such soil shall be replaced by compacted select fill.

10.6.1.7 In freezing-and-thawing zones, the select fill shall be a non-frost-susceptible crushed granular fill.

10.6.2 *Foundation heating*

10.6.2.1 *In temperate climates*—that is, areas where there is no permafrost—foundations in contact with the soil shall require a heating system or other method to prevent the 32°F isotherm from penetrating the soil and causing frost heave.

R10.6.2.1 The soil beneath a tank bearing on the ground is prone to losing heat to the tank, and this may lead to freezing of the ground and cause frost heave in temperate climates. Controlling the position of the 32°F isotherm prevents freezing the soil below the tank that can cause frost-heave forces on the base of the tank. Frost heave may be avoided by trace heating the base slab or elevating the base slab, allowing heat input to the foundation through natural air convection.

Air gaps under refrigerated liquefied gas (RLG) tanks are effective to prevent ground freezing instead of foundation heating systems. Heating systems are not required with elevated foundations, having an air gap that prevents ground freezing due to stored RLG. The open height should be sufficient to ensure good airflow even at the end of design life considering long-term settlement.

In areas of permafrost, heating systems will generally not be used and are likely to be detrimental. Ice is not always present, as may be the case of nonporous bedrock, but it frequently occurs and may be in amounts exceeding the potential hydraulic saturation of the ground material. Overlying permafrost is a thin active layer that seasonally thaws during the summer. As in temperate zones, designers often try to maintain the temperature regime in the ground that

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existed before site disturbance. Designers should always consult geotechnical engineers knowledgeable in permafrost behavior when building on permafrost.

The designer should consider the risk of a gap under an elevated tank filling with flammable vapor, liquid, or both, in case of a tank leak or a leak from adjacent piping. The air gap space should be designed so that vapor is not trapped in a confined space. The designer should consider U.S. Occupational Safety and Health Administration (OSHA) Standard 29 CFR 1910.146 for confined space entry. The air gap space of individual tanks may or may not qualify as a confined space.

The designer should consider the prevailing winds and airflow in the tank area. The opening for airflow should be sufficient to keep the vapor concentration below 1/2 of the lower explosive limit. Calculations should be made to show that the air gap space and ventilation are sufficient to show that the flame propagation speed is below the speed necessary to cause overpressurization and explosion.

Deflectors may be used to increase airflow under the tank. The air gap should be sufficient to allow adequate airflow and reasonable access under the tank for monitoring and cleaning purposes. The dimensions of the space may be adjusted on agreement of the owner and engineer. The space should remain drained and dry during normal operations.

10.6.2.2 Heating systems shall be designed to allow functional and performance monitoring.

R10.6.2.2 In designing a heating system and selecting the bearing depth of the foundation, consideration should be given to the potential for frost heave due to natural freezing of the soil before the heating system is activated. The foundation should bear at a depth that is below the shrinking-and-swelling or freezing-and-thawing zone, or the bearing material should be selected to be unaffected by temperature or moisture changes.

10.6.2.3 Details of the heating system shall include provisions for:

a) Individual replacement of any heating element or temperature sensor

b) Protection against ingress of water and moisture that can cause galvanic corrosion or other forms of deterioration

R10.6.2.3 The heating system should be designed to allow maintenance such as replacing the heating elements or thermal sensors on a routine, in-service basis. Functional and performance monitoring should be performed on a weekly basis as a minimum frequency. Naturally occurring clean coarse sand or gravel will not be susceptible to frost heave as long as it is well drained.

10.7—Foundation performance monitoring details

10.7.1 Tank foundations shall be monitored and recorded for settlement before, during, and after the hydrotest in accordance with 10.8.2.1. When settlement monitoring indicates settlement exceeding 75 percent of predefined values, the engineer shall be notified.

R10.7—Foundation performance monitoring details

R10.7.1 Baseline elevations of the tank foundation should be established prior to hydrotesting. Settlement at tank center is measured remotely with inclinometers discussed in 10.7.3 or, in the case of hydrotest, by entering the tank and making a level survey after testing.

As a minimum for tanks where settlement predictions indicate values expected near the limiting values, or wherever the owner specifies, two conduits arranged orthogonally should be cast into the foundation to accommodate settlement-measuring instrumentation, such as inclinometers, and thus provide settlement profiles across the tank bottom.

During hydrotesting of large tanks, settlement measurements should be made after water filling has reached levels 1/4, 1/2, 3/4, and full. The hydrotest procedure should include the permissible values of settlement for each level of filling for comparison during testing. Differential settlement and tilting should be compared with permissible values at each level with review by the design team if results approach the permissible values. During emptying the hydrotest water, settlement measurements should be made to measure rebound after emptying has reached the 3/4, 1/2, 1/4, and empty levels as a minimum. API 620, Appendix C, provides additional guidance on settlement. API 620, Section Q.8.4, discusses settlement measurements. API 620, Appendix R, discusses settlement measurements in several sections.

10.7.2 *Survey points and benchmarks*

10.7.2.1 A minimum of eight permanent survey points for measuring elevation shall be installed at equal intervals around the periphery of the tank foundation.

R10.7.2.1 The number of installed survey points should be divisible by four. This allows easy examination of planar cross sections through the tank and comparison with geologic cross sections obtained during the geotechnical investigation.

10.7.2.2 Spacing between adjacent survey points shall not exceed 33 ft.

10.7.2.3 The survey points shall be referenced to at least one external permanent benchmark.

R10.7.2.3 External permanent benchmarks should be tied into the international terrestrial reference frame system (ITRF) with a precision of ±0.01 ft to provide precise recovery should the permanent benchmark be destroyed or subject to regional settlement.

10.7.2.4 Upon foundation completion and before wall construction, permanent survey points shall be installed and their locations documented.

10.7.3 *Inclinometers*—Inclinometers shall be installed in the foundation for site classes other than Site Class A (hard rock) or Site Class B (firm rock) as defined in ASCE/SEI 7.

R10.7.3 Baseline elevations of the tank foundation should be established prior to hydrotesting. Settlement at tank center is measured remotely with inclinometers discussed in 10.7.3 or, in the case of hydrotest, by entering the tank and making a level survey after testing.

As a minimum for tanks where settlement predictions indicate values expected near the limiting values, or wherever the owner specifies, two conduits arranged orthogonally should be cast into the foundation to accommodate settlement-measuring instrumentation, such as inclinometers, and thus provide settlement profiles across the tank bottom.

10.7.4 *Thermal monitoring*

10.7.4.1 A thermal monitoring system shall be installed in the foundation to monitor the temperature of the foundation to assess the performance of bottom insulation heating system.

10.7.4.2 The thermal monitoring system shall be monitored to detect adverse cryogenic effects on the ground below the foundation.