Design and Construction of Circular Wire- and Strand-Wrapped Prestressed Concrete Structures

Reported by ACI Committee 372

Andrew E. Tripp, Jr.
Chair

Nicholas A. Legatos
Secretary

Jon B. Ardahl
Richard L. Bice
Ashok K. Dhingra
Charles S. Hanskat
Frank J. Heger
William J. Hendrickson
Salvatore Marques
William C. Schnobrich
Morris Schupack
Marwan N. Youssef

This report provides recommendations for the design and construction of wrapped, circular, prestressed concrete structures commonly used for liquid or bulk storage. These structures are constructed using thin cylindrical shells of either concrete or shotcrete. Shotcrete and precast concrete core walls incorporate a thin steel diaphragm that serves both as a liquid barrier and vertical reinforcement. Cast-in-place concrete core walls incorporate either vertical prestressing or a steel diaphragm. Recommendations are given for circumferential prestressing achieved by wire or strand wrapping. In wrapping, the wire or strand is fully tensioned before placing it on the structural core wall. Procedures for preventing corrosion of the prestressing elements are emphasized. The design and construction of dome roofs are also covered.

Many recommendations of this report can also be applied to similar structures containing low-pressure gases, dry materials, chemicals, or other materials capable of creating outward pressures. This report is not intended for application to nuclear reactor pressure vessels or cryogenic containment structures.

Keywords: circumferential prestressing; dome; footing; joint; joint sealant; prestressed concrete; prestressing steel; shotcrete; wall.

CONTENTS

ACI 372R-03 supersedes ACI 372R-00 and became effective June 18, 2003.

CHAPTER 1—GENERAL

1.1—Introduction

The design and construction of circular prestressed concrete structures requires specialized engineering knowledge and experience. The recommendations herein reflect over five decades of experience in designing and constructing circular prestressed structures. When designed and built with understanding and care, these structures can be expected to serve for well over 50 years without requiring significant maintenance.

1.2—Objective

This report provides guidance for individuals responsible for the design and construction of circular prestressed concrete structures by recommending practices used in successful structures.

1.3—Scope

The recommendations supplement the general requirements for reinforced concrete and prestressed concrete design and construction given in ACI 318-99, ACI 350-01, and ACI 301. Design and construction recommendations cover the following elements or components of circular-wrapped prestressed concrete structures:

I. *Floors*
  Reinforced concrete
II. *Floor-wall connections*
  Hinged;
  Fixed;
  Partially fixed;
  Unrestrained; and
  Changing restraint.
III. *Walls*
  Cast-in-place concrete walls with steel diaphragms or vertical prestressing;
  Shotcrete walls with steel diaphragms; and
  Precast concrete walls with steel diaphragms.
IV. *Wall-roof connections*
  Hinged;
  Fixed;
  Partially fixed; and
  Unrestrained.
V. *Roofs*
  Concrete dome roofs with prestressed dome ring, constructed with cast-in-place concrete, shotcrete, or precast concrete; and
  Flat concrete roofs.
VI. *Wall and dome ring prestressing systems*
  Circumferential prestressing using wrapped wire or strand systems; and
  Vertical prestressing using single or multiple high-strength strands, bars, or wires.

1.4—Associated structures

The following types of structures are frequently constructed inside water storage tanks:

- Baffle walls; and
- Inner storage walls.

Baffle walls are used to increase the chlorine retention time (CT) of water as it circulates from the tank inlet to the outlet. The configuration and layout of baffle walls vary depending on the tank geometry, flow characteristics, and the desired effectiveness of the chlorination process. The most common baffle wall configurations are straight, C-shaped, or a combination of the two. Baffle walls can be precast or cast-in-place concrete, masonry block, redwood, shotcrete, metal, or fabric.

Inner storage walls are separate storage cells normally used to provide flexibility in a system's water storage capabilities and hydraulics. Inner walls are typically constructed the same as the outer tank walls and are designed for external and internal hydrostatic pressure.

1.5—History and development

The first effort to apply circumferential prestressing to a concrete water tank is attributed to W. S. Hewett, who, in the early 1920s, used turnbuckles to connect and tension individual steel tie rods. Long-term results were not effective because the steel used was of low yield strength, limiting applied unit tension to approximately 30,000 lb/in.^{2} (210 MPa). Shrinkage and creep of the concrete resulted in a rapid and almost total loss of the initial prestressing force. Eugene Freyssinet, the distinguished French engineer regarded as the father of prestressed concrete, was the first to realize the need to use steel of high quality and strength, stressed to relatively high levels, to overcome the adverse effects of concrete creep and shrinkage. Freyssinet successfully applied prestressing to concrete structures as early as the late 1920s. Vertical wall prestressing was introduced in the 1930s as a means to control horizontal cracking that might permit leakage and subsequent corrosion of circumferential prestressing steel.

In 1942, J. M. Crom, Sr. (the first to apply high-strength prestressing steels to concrete tanks), developed a novel method to apply high-strength wire in a continuous spiral to the exterior surface of concrete tanks. The method is based on mechanically stressing the wire as it is placed on the wall, thus avoiding prestressing loss due to friction between the prestressed reinforcement and the wall. This method of circumferentially prestressing tank walls and dome rings is commonly known as wire winding or wire wrapping. After placement, the prestressed reinforcement is protected from corrosion by encasing it in shotcrete. More than 6000 tanks of various sizes and shapes have been constructed using methods based on this concept.

In 1952, shotcrete tanks incorporating a light-gage steel diaphragm fluid barrier (Section 2.3.2.1.3) within the wall were first built by J. M. Crom, Sr.; by the early 1960s, nearly all prestressed shotcrete tanks used a steel diaphragm. In 1966, the first precast-prestressed concrete tanks with a steel diaphragm were built. By 1970, nearly all wire-wound precast concrete tanks incorporated a steel diaphragm or, alternatively, vertical prestressing within the wall. The use of a steel diaphragm or vertical prestressing prevents the stored liquid from penetrating to the outside of the core wall where it could potentially contribute to the corrosion of the prestressing steel. The diaphragm also serves as vertical reinforcement.

1.6—Definitions

Definitions used in this report are in addition to those included in ACI 318-99.

Anchorage—In post-tensioning, a device used to anchor the tendon to the concrete member; in pretensioning, a device used to anchor the tendon during hardening of concrete. Note: The anchorage transfers the tensile force from the tendon into the concrete.

Body coat—The layers of shotcrete applied over the outermost wire coat, not in direct contact with prestressing wire or strand.

Bonded tendon—A prestressing tendon that is bonded to concrete either directly or through grouting.

Breathable or breathing-type coating—coating that permits transmission of water vapor without detrimental effects to the coating.

Changing restraint—A joint of a different type during and after prestressing. Note: An example is a joint unrestrained during prestressing then hinged after prestressing; the change in joint characteristics results from the grout installation after prestressing that prevents further radial translation.

Core wall—That portion of a concrete wall that is circumferentially prestressed.

Fixed—Full restraint of radial translation and full restraint of rotation.

Hinged—Full restraint of radial translation and negligible restraint of rotation.

Joint restraint conditions—Top and bottom boundary conditions for the cylindrical shell wall or the dome edge.

Membrane floor—A thin, highly reinforced, cast-in-place slab-on-ground designed to deflect when the subgrade settles and still retain watertightness.

Partially fixed—Full restraint of radial translation and partial restraint of rotation.

Shotcrete cover coat—Shotcrete covering the outermost layer of wrapped prestressing strand or wire, usually consisting of the wire coat plus the body coat.

Tank—A structure commonly used for liquid or bulk storage. As used in this document, the term tank refers to a circular wire- or strand-wrapped prestressed concrete structure.

Tendon—A steel element, such as wire, bar, cable or strand, or a bundle of such elements, used to impart prestress to concrete. Note: In pretensioned concrete, the tendon is the steel element. In post-tensioned concrete, the tendon includes end anchorages, couplers, or both; prestressing steel; and sheathing filled with portland-cement grout, grease, or epoxy grout.

Wire coat—The layer of shotcrete in direct contact with the prestressing wire or strand.

Wrapped prestressing—A prestressing system using wire or strand that is fully tensioned before placement on the core wall.

1.7—Notation

A_g = gross area of unit height of core wall that resists circumferential force, in.^{2} (mm^2)

A_{gr} = gross area of wall that resists externally applied circumferential forces, such as backfill, in.^{2} (mm^2)

A_{ps} = area of prestressed circumferential reinforcement, in.^{2} (mm^2)

A_s = area of nonprestressed circumferential reinforcement, in.^{2} (mm^2)

B_c = buckling reduction factor for creep, nonlinearity, and cracking of concrete

B_i = buckling reduction factor for geometrical imperfection

D = dead loads or related internal moments and forces

E_c = modulus of elasticity of concrete under short-term load, lb/in.^{2} (MPa)

E_s = modulus of elasticity of steel, lb/in.^{2} (MPa)

f'_c = specified compressive strength of concrete, lb/in.^{2} (MPa)

f'_{ci} = compressive strength of concrete at time of prestressing, lb/in.^{2} (MPa)

f'_g = specified compressive strength of shotcrete, lb/in.^{2} (MPa)

f'_{gi} = compressive strength of shotcrete at time of prestressing, lb/in.^{2} (MPa)

f_{pu} = specified tensile strength of prestressing wires or strands, lb/in.^{2} (MPa)

f_y = specified yield strength of nonprestressed reinforcement, lb/in.^{2} (MPa)

h = wall thickness, in. (mm)

h_d = dome shell thickness, in. (mm)

L = live loads lb/ft^2 (kPa)

n = modular ratio of elasticity = E_s/E_c

P_e = circumferential force per unit of height of wall caused by the effective prestressing, lb (N)

P_h = circumferential force per unit of height of wall caused by the external pressure of soil, groundwater, or other loads, lb (N)

P_o = nominal axial compressive strength of core wall in the circumferential direction per unit of height of wall, lb/in.^{2} (MPa)

p_u = factored design load on dome shell, lb/ft^2 (kPa)

r = inside radius of tank, ft (mm)

r_d = mean radius of dome, ft (mm)

r_i = averaged maximum radius of curvature over a dome imperfection area with a diameter of $2.5\sqrt{r_d h_d/12}$ ft ($2.5\sqrt{r_d h_d}$ [mm])
t = floor slab thickness, in. (mm)
y = differential floor settlement (between outer perimeter and tank center), in. (mm)
ϕ = strength-reduction factor

Notes:
A. The inch-pound units are the primary units used in the text. SI conversions are hard conversions of the inch-pound values and are shown in parenthesis.
B. Coefficients in equations that contain $\sqrt{f'_c}$ or $\sqrt{f'_g}$ are based on inch-pound (lb/in.^{2}) units. The coefficients to be used with $\sqrt{f'_c}$ and $\sqrt{f'_g}$ in the SI (MPa) system are the inch-pound coefficients divided by 12.

CHAPTER 2—DESIGN

2.1—Strength and serviceability

2.1.1 *General*—Structures and their components should be designed to meet both the minimum strength and serviceability recommendations contained in this report. These recommendations are intended to provide adequate safety and performance of structures subject to typical loads and environmental conditions. Controlling leakage and protection of all embedded steel from corrosion is necessary for adequate serviceability.

2.1.2 *Loads and environmental conditions*

2.1.2.1 The following loads, forces, and pressures should be considered in the design:

- Prestressing forces—circumferential prestressing forces in the walls and dome rings; vertical prestressing, if used in the walls; and roof prestressing if used;
- Internal pressure from stored materials, such as fluid pressure in liquid storage vessels, gas pressure in vessels containing gas or materials that generate pressure, and lateral pressure from stored granular materials. For pressure from stored granular materials, refer to ACI 313;
- External lateral earth pressure, including the surcharge effects of live loads supported by the earth acting on the wall;
- Weight of the structure;
- Wind load;
- Snow and other imposed loads (earth where applicable) on roofs;
- Hydrostatic pressure on walls and floors due to groundwater;
- Seismic effects; and
- Equipment and piping supported on roofs or walls.

2.1.2.2 In addition to those listed in Section 2.1.2.1, the following effects should also be considered:

- Loss of prestressing force due to concrete and shotcrete creep and shrinkage, and relaxation of prestressing steel;
- Temperature and moisture differences between structural elements;
- Thermal and moisture gradients through the thickness of structural elements;
- Exposure to freezing-and-thawing cycles;
- Chemical attack on concrete and metal; and
- Differential settlements.

2.1.2.3 One or more of the following means should be used, whenever applicable, to prevent the design loads from being exceeded:

- Positive means, such as an overflow pipe of adequate size, should be provided to prevent overfilling liquid-containment structures. Overflow pipes, including their inlet and outlet details, should be capable of discharging the liquid at a rate equal to the maximum fill rate when the liquid level in the tank is at its highest acceptable level.
- One or more vents should be provided for liquid and granular containment structures. The vent(s) should limit the positive internal pressure to an acceptable value when the tank is being filled at its maximum rate and limit the negative internal pressure to an acceptable value when the tank is being emptied at its maximum rate. For liquid-containment structures, the maximum emptying rate may be taken as the rate caused by the largest tank pipe being broken immediately outside the tank.
- Hydraulic pressure-relief valves can be used on nonpotable water tanks to control hydrostatic uplift on slabs and the hydrostatic pressure on walls when the tanks are empty or partially full. The use of pressure-relief valves should be restricted to applications where the expected groundwater level is below the operating level of the tank. The valves may also be used to protect the structure during floods. A sufficient number of valves should be used to provide at least 50% system redundancy. No fewer than two valves should be used, with at least one valve being redundant. The inlet side of the pressure-relief valves should be interconnected with:

(a) A layer of free-draining gravel adjacent to and underneath the concrete surface to be protected;

(b) A perforated-type drain system placed in a free-draining gravel adjacent to and underneath the concrete surface to be protected; or

(c) A perforated pipe drain system in a free-draining gravel that serves as a collector system for a geomembrane drain system placed against the concrete surface to be protected.

The pressure-relief-valve inlet should be protected against the intrusion of gravel by a corrosion-resistant screen; an internal corrosion-resistant strainer; or by a connected, perforated pipe drain system. The free-draining gravel interconnected with the pressure relief valves should be protected against the intrusion of fine material by a sand filter or geotextile filter.

The spacing and size of pressure-relief valves should be adequate to control the hydrostatic pressure on the structure, and the valves should not be less than 4 in. (100 mm) in diameter and should not be spaced more than 20 ft (6 m) apart. Some or all valves should be placed at the lowest part of the structure, unless the structure has been designed to withstand the pressure imposed by a groundwater level to, or slightly above, the elevation of the valves. The use of spring-controlled, pressure-relief valves is discouraged, as they may be prone to malfunction of the springs. The recommended pressure-relief valves are:

1. Floor-type pressure-relief valves that operate by hydrostatic pressure lifting a cover where travel is limited by restraining lugs; and

2. Wall-type pressure-relief valves with corrosion-resistant hinges operated by hydrostatic pressure against a flap gate.

When using floor-type valves, note that operation can be affected by sedimentation within the tank, incidental contact by a scraper mechanism in the tank, or both. When wall-type valves are used in tanks with scraper mechanisms, the valves should be placed to clear the operating scraping mechanisms with the flap gate in any position, taking into account that there can be some increase in elevation of the mechanisms due to buoyancy, buildup of sediment on the floor of the tank, or both.

Gas pressure-relief valves should be used to limit gas pressure to an acceptable level on the roof and walls on non-vented structures, such as digester tanks. The pressure-relief valve should be compatible with the anticipated contained gas and the pressure range. The valve selection should consider any test pressure that may be required for the structure.

2.1.3 *Strength*

2.1.3.1 *General*—Structures and structural members should be proportioned to have design strengths at all sections equal to or exceeding the minimum required strengths calculated for the factored loads and forces in such combinations as required in ACI 318-99 and as recommended in this report.

2.1.3.2 *Required strength*—The load factors required in ACI 318-99 for dead load, live load, wind load, seismic forces, and lateral earth pressure should be used. A load factor of 1.4 should be used for liquid and gas pressure, with the exception that the load factor for gas pressure can be reduced to 1.25 for domes with pressure-relief valves. A load factor of 1.4 should be applied to the final effective prestressing forces for determining the required circumferential strength of the core wall. When prestressing restraint moments, in combination with other factored loads and environmental effects produce the maximum flexural strength requirements, a load factor of 1.2 should be applied to the maximum applicable initial or final prestressing force. When prestressing restraint moments reduce the flexural strength required to resist other factored loads and environmental effects, a load factor of 0.9 should be applied to the minimum applicable prestressing force. Refer to ACI 313 for load factors for lateral pressures from stored granular materials. To design structural floors for hydrostatic uplift, a load factor of 1.5 should be applied to the hydrostatic uplift forces.

2.1.3.3 *Design strength*—The design strength of a member or cross section should be taken as the product of the nominal strength, calculated in accordance with the provisions of ACI 318-99, multiplied by the applicable strength reduction factor, except as modified in this report.

The strength-reduction factor should be as required in ACI 318-99, except as follows:

- Tension in circumferential prestressed reinforcement, $\phi = 0.85$; and
- Circumferential compression in concrete and shotcrete, $\phi = 0.75$.

A strength check need not be made for initial prestressing forces that comply with provisions of Section 2.3.3.2.1.

2.1.4 *Serviceability recommendations*

2.1.4.1 *Liquid-tightness control*—Liquid-containing structures should not exhibit visible flow or leakage as defined in Section 5.3. Acceptance criteria for liquid-tightness are given in Chapter 5.

2.1.4.2 *Corrosion protection of prestressed reinforcement*—Circumferential prestressed wire or strand placed on the exterior surface of a core wall or a dome ring should be protected by at least 1 in. (25 mm) of shotcrete cover. Each wire or strand should be encased in shotcrete. Vertical prestressed reinforcement should be protected by portland cement or epoxy grout. The requirements for concrete protection of vertical tendon systems and minimum duct and grout requirements are given in ACI 318-99.

2.1.4.3 *Corrosion protection of nonprestressed reinforcement*—Nonprestressed reinforcement should be protected by the amount of concrete cover as required in ACI 350-01 and summarized as follows:

(a) **Floor slabs**	**Minimum cover, in. (mm)**
From top of slab	
Membrane slabs ($t \leq 6$ in.)	1 (25)
Slabs-on-ground ($t \leq 8$ in.)	1-1/2 (40)
Structural slabs-on-ground more than 8 in. thick	2 (50)
From slab underside	
Membrane slabs ($t \leq 6$ in.) and Slabs-on-ground ($t \leq 8$ in.):	
Slabs cast against a stabilized subgrade or plastic vapor barrier	1-1/2 (40)
Slabs cast against a non-stabilized subgrade or without vapor barrier	2 (50)
Slabs more than 8 in. thick (regardless of subgrade condition—except as provided for ACI 350-01, R7.7, and Section 1.4)	3 (75)
(b) **Wall**	
From inside face	1 (25)
From outside face (over steel diaphragm)	1 (25)
(c) **Dome roof**	
From top surface	1 (25)
From roof underside	1 (25)
(d) **Flat roof**	
From top surface	2 (50)
From roof underside	2 (50)

2.1.4.4 *Boundary conditions*—The effects of translation, rotation, and other deformations should be considered. The effects originating from prestressing, loads, and volume changes, such as those produced by thermal and moisture changes, concrete creep, and relaxation of prestressed reinforcement, should also be considered.

2.1.4.5 *Other serviceability recommendations for liquid-containing structures*—Allowable stresses, provisions for determining prestressing losses, recommendations for liquid barriers or bidirectional prestressing to preclude leakage, and various other design recommendations intended to ensure serviceability of water tanks and other liquid-containing structures are given in Sections 2.2 to 2.4.

2.2—Floor and footing design

2.2.1 *Foundations*—Refer to Appendix A for recommendations and considerations related to the design and construction of tank foundations.

2.2.2 *Membrane floor slabs*—Membrane floor slabs transmit loads directly to the subbase without distribution. Settlements should be anticipated and provisions made for their effects. Local hard and soft spots beneath the floor should be avoided or considered in the floor design.

2.2.2.1 The minimum thickness of membrane floor slabs should be 4 in. (100 mm).

2.2.2.2 To limit crack widths and spacing, the minimum ratio of reinforcement area to concrete area should be 0.005 in each horizontal orthogonal direction, except as recommended in Section 2.2.2.7.

2.2.2.3 Additional reinforcement should be provided at floor edges and other discontinuities as required by the connection design. In tanks with hinged or fixed base walls, additional reinforcement should be provided as required in the edge region to accommodate tension in the floor slab caused by the radial shear forces and bending moments induced by restraint at the wall base.

2.2.2.4 In cases of restraint to floor movement, such as large underfloor pipe encasements, details to limit crack width and spacing should be provided.

Details used successfully include gradual transitions in thickness between pipe encasements and floors, separating pipe encasements from floors through the use of horizontal joints, and the use of additional reinforcement in pipe encasements not separated from floors.

2.2.2.5 Reinforcement should be either welded-wire fabric or deformed bar. Maximum-wire spacing for welded-wire fabric should be 4 in. (100 mm), and adjacent sheets or rolls of fabric should be overlapped a minimum of 6 in. (150 mm). Maximum spacing of bar reinforcement should be 12 in. (300 mm). These maximum spacings provide crack control.

2.2.2.6 Reinforcement should be located in the upper 2-1/2 in. (65 mm) of the slab thickness, with the minimum covers recommended in Section 2.1.4.3, and should be maintained at the correct elevation by support chairs or concrete cubes.

2.2.2.7 Slabs greater than 8 in. (200 mm) thick should have a minimum reinforcement ratio of 0.006 in each orthogonal direction and distributed into two mats of reinforcing steel. One mat should be located in the upper 2-1/2 in. (65 mm) of the slab thickness and should provide a minimum ratio of reinforcement area to total concrete area of 0.004 in each orthogonal direction. The second mat should be located in the lower 3-1/2 in. (90 mm) of the slab and provide a minimum ratio of reinforcement area to total concrete area of 0.002 in each orthogonal direction. Minimum covers from the reinforcing steel mats to the top of the slab and the underside should be as recommended in Section 2.1.4.3. Slabs thicker than 24 in. (600 mm) need not have reinforcement greater than that required for a 24 in. (300 mm) thick slab. In wall footings monolithic with the floor, the minimum ratio of circumferential reinforcement area to concrete area should be 0.005.

2.2.2.8 A floor subjected to hydrostatic uplift pressures that exceed 0.67 times the weight of the floor should be provided with subdrains or pressure-relief valves to control uplift pressures or be designed as structural floors in accordance with the recommendations given in Section 2.2.3. Pressure-relief valves will allow contamination of the tank contents by groundwater or contamination of the subgrade by untreated tank contents.

2.2.3 *Structural floors*—Structural floors should be designed in accordance with ACI 350-01. Structural floors are required when piles or piers are used because of inadequate soil-bearing capacity, hydrostatic uplift, or expansive subgrade. Structural floors can also be used where excessive localized soil settlements reduce support of the floor slab, such as where there is a potential for sinkholes.

2.2.4 *Mass concrete*—Concrete floors used to counteract hydrostatic uplift pressures can be mass concrete as defined in ACI 116R and ACI 207.1R. Minimum reinforcement recommendations are given in Section 2.2.2.7. The effect of restraint, volume change, and reinforcement on cracking of mass concrete is the subject of ACI 207.2R.

2.2.5 *Floor concrete strength*—Minimum concrete strength recommendations are given in Section 3.1.4.

2.2.6 *Floor joints*—For liquid-containing structures, membrane floors should be designed so that the entire floor can be cast without cold joints or construction joints. If this is not practical, the floor should be designed to minimize construction joints.

2.2.7 *Wall footing*

2.2.7.1 A footing should be provided at the base of the wall to distribute vertical and horizontal loads to the subbase or other support. The footing may be integral with the wall, floor, or both.

2.2.7.2 Recommendations for spacing and minimum ratio for circumferential reinforcement are given in Sections 2.2.2.5 and 2.2.2.7, respectively.

2.2.7.3 The bottoms of footings on the perimeter of a tank should extend at least 12 in. (300 mm) below the adjacent finished grade. A greater depth may be required for frost protection or for adequate soil bearing.

2.3—Wall design

2.3.1 *Design methods*—The design of the wall should be based on elastic cylindrical shell analyses considering the effects of prestressing, internal loads, backfill, and other external loads. The design should also provide for:

- The effects of shrinkage, elastic shortening, creep, relaxation of prestressed reinforcement, and temperature and moisture gradients;
- The joint movements and forces resulting from the restraint of deflections, rotations, and deformations that

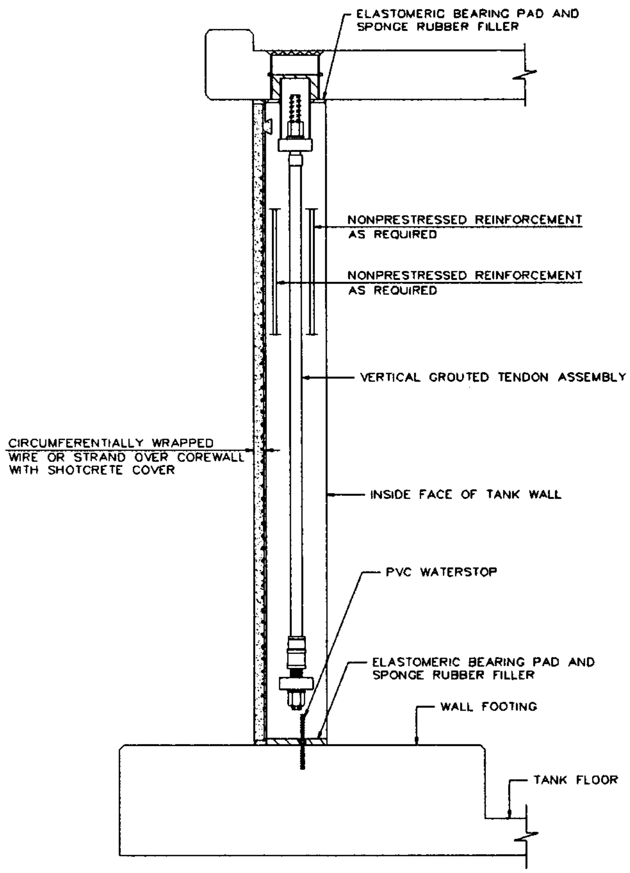

Fig. 2.1—Typical wall section of a wire- or strand-wrapped, cast-in-place, vertically prestressed tank.

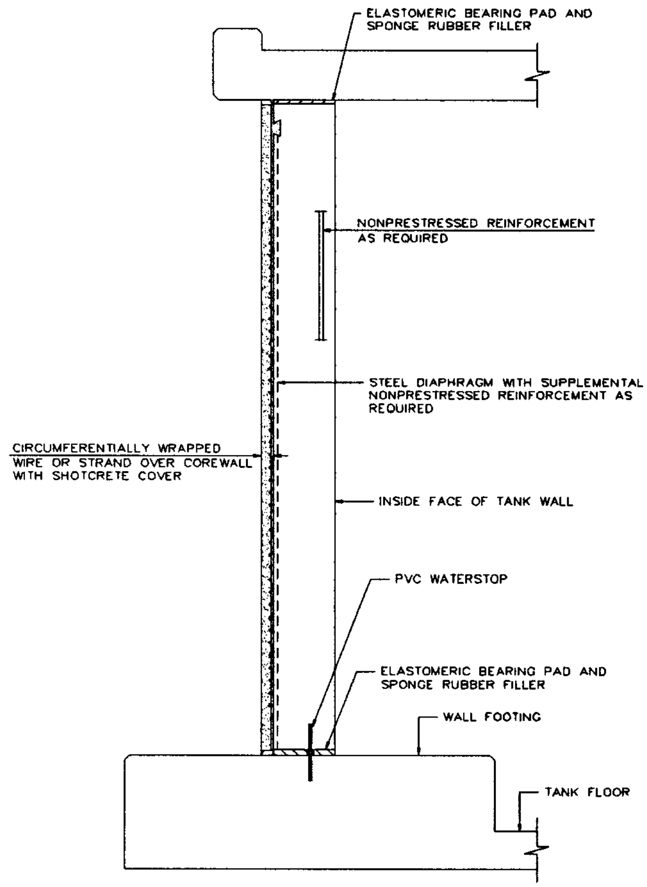

Fig. 2.2—Typical wall section of a wire- or strand-wrapped, cast-in-place tank with a steel diaphragm.

are induced by prestressing forces, design loads, and volume changes; and

- Thermal stresses. These stresses are often evaluated using inelastic methods of analysis, which usually involve the use of a reduced modulus of elasticity.[1]

Coefficients, formulas, and other aids (based on elastic shell analyses) for determining vertical bending moments, and circumferential, axial, and radial shear forces in walls are given in References 2 through 8.

2.3.2 *Wall types*—This report describes four wall types used in liquid-containing structures:

2.3.2.1.1 *Cast-in-place concrete, prestressed circumferentially by wrapping with either high-strength steel wire or strand, wound on the external surface of the core wall, and prestressed vertically with grouted steel tendons*—Vertical nonprestressed steel reinforcement may be provided near each face for strength and to limit crack width and spacing. Nonprestressed temperature reinforcement should be considered in situations where the core wall is subject to significant temperature variations or shrinkage before circumferential or vertical prestressing is applied. The circumferential prestressing is encased in shotcrete that provides corrosion protection and bonding to the core wall (Fig. 2.1).

2.3.2.1.2 *Cast-in-place concrete with full-height, vertically fluted steel diaphragm, prestressed circumferentially by wrapping with either high-strength steel wire or strand*—The steel diaphragm is located on the exterior face and the vertical steel reinforcement near the interior face. Adjacent sections of the diaphragm are joined and sealed, as recommended in Section 3.7.1, to form an impervious membrane. The exposed diaphragm is coated first with shotcrete, after which the composite wall is prestressed circumferentially by winding with high-strength wire or strand. Grouted post-tensioned tendons can be provided for vertical reinforcement. The circumferential prestressing is encased in shotcrete that provides corrosion protection and bonding to the core wall (Fig. 2.2).

2.3.2.1.3 *Shotcrete with full-height vertically fluted steel diaphragm, prestressed circumferentially by wrapping with either high-strength steel wire or strand*— Diaphragm steel is provided near one face, and nonprestressed steel reinforcement is provided near the other face as vertical reinforcement. If needed, additional nonprestressed steel can be provided in the vertical direction near the face with the diaphragm. Adjacent sections of the diaphragm are joined and sealed, as recommended in Section 3.7.1, to form an impervious membrane. Grouted post-tensioned tendons can be provided as vertical reinforcement. The circumferential prestressing is encased in shotcrete that provides corrosion protection and bonding to the core wall (Fig. 2.3).

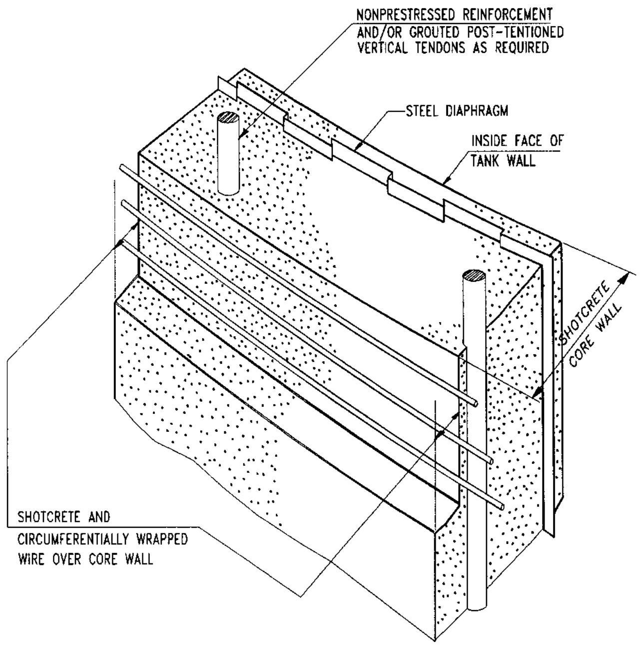

Fig. 2.3—Typical wall section of wire- or strand-wrapped shotcrete tank with steel diaphragm.

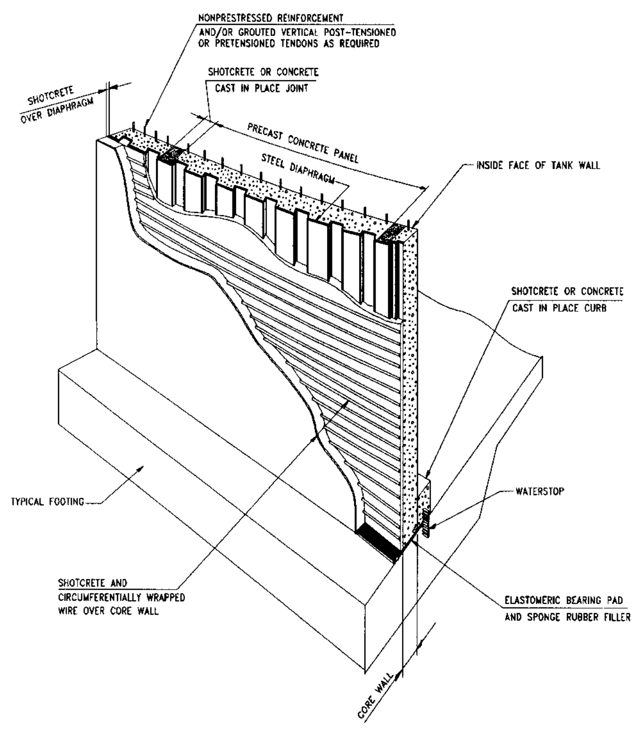

Fig. 2.4—Typical wall section of wire- or stand-wrapped precast tank with steel diaphragm.

2.3.2.1.4 *Precast concrete vertical panels curved to tank radius with a full-height, vertically fluted steel diaphragm prestressed circumferentially by wrapping with either high-strength steel wire or strand*—The vertical panels are connected with sheet steel, and the joints between the panels are filled with cast-in-place concrete, cement-sand mortar, or shotcrete. Adjacent sections of the diaphragm, both within the panels and between the panels, are joined and sealed, as recommended in Section 3.7.1, to form a solid membrane. The exposed diaphragm is coated first with shotcrete, after which the composite wall is prestressed circumferentially by winding with high-strength steel wire or strand. Grouted post-tensioned or pretensioned tendons may be provided for vertical reinforcement. The circumferential prestressing is encased in shotcrete that provides corrosion protection and bonding to the core wall (Fig. 2.4).

2.3.2.2 *Liquid-tightness*—In a shotcrete, cast-in-place, or precast concrete wall, liquid-tightness is achieved by the circumferential prestressing and by a liquid-tight steel diaphragm incorporated into the core wall. A cast-in-place wall can also achieve liquid-tightness by using both circumferential and vertical prestressed reinforcement. Considerations of special importance with respect to liquid-tightness are:

- A full height, vertically fluted steel diaphragm with sealed edge joints that extends throughout the wall area provides a positive means of achieving liquid-tightness;
- Vertical prestressing, in cast-in-place core walls without a diaphragm, provides a positive means of limiting horizontal crack width, thus providing liquid-tightness;
- Circumferential (horizontal) construction joints between the wall base and the top should not be permitted in the core wall; only the wall base joint and vertical joints should be permitted. The necessity of obtaining concrete free of honeycombing and cold joints cannot be overemphasized; and
- All vertical construction joints in cast-in-place concrete core walls without a metal diaphragm should contain waterstops and dowels to prevent radial displacement of adjacent wall sections.

2.3.3 *Wall proportions*

2.3.3.1 *Minimum core wall thickness*—Experience in wrapped prestressed tank design and construction has shown that the minimum core wall thickness should be as follows:

- 7 in. (180 mm) for cast-in-place concrete walls;
- 3-1/2 in. (90 mm) for shotcrete walls with a steel diaphragm; and
- 4 in. (100 mm) for precast-concrete walls with a steel diaphragm.

2.3.3.2 *Circumferential compressive stress*

2.3.3.2.1 *Maximum stress at initial prestressing*—The circumferential compressive stress in the core wall produced by the unfactored initial prestressing force should not exceed $0.55f'_{ci}$ for concrete and $0.55f'_{gi}$ for shotcrete. The stress should be determined based on the net core wall area after deducting all openings, ducts, and recesses, including the effects of diaphragm joints.

Experience with the previously mentioned maximum initial compressive stress is limited to a maximum design concrete strength, f'_c, of 5000 lb/in.^{2} (35 MPa), and shotcrete strength, f'_g of 4500 lb/in.^{2} (31 MPa). Caution is advised if higher compressive-strength concrete is used. If higher concrete strengths are used, additional design considerations, such as buckling and stability, should be investigated.

2.3.3.2.2 *Resistance to final prestressing*—The compressive strength of any unit height of wall for resisting final circumferential prestressing force (after all losses recommended in the following) should be

$$0.85 f'_c \phi [A_g + (2n-1)A_s] \geq 1.4 P_e \text{ lb (N)} \qquad (2\text{-}1)$$

(use f'_g if shotcrete)

2.3.3.2.3 *Resistance to external load effects*—For resisting factored external load effects, such as backfill, the compressive strength of any unit height of wall should be the compressive strength of the wall reduced by the core wall strength required to resist 1.4 times the final circumferential prestressing force.

$$\phi(0.85 f'_c A_{gr} + A_s f_y)\left\{1 - \frac{1.4 P_e}{0.85 f'_c \phi [A_g + (2n-1)A_s]}\right\}$$

$$\geq 1.7 P_h \text{ lb (or N in the SI system)} \qquad (2.2)$$

(use f'_g if shotcrete)

2.3.3.2.4 *Compressive strain limit*—The wall should be proportioned so that the compressive axial strain remains within the elastic range under the effects of prestressing plus other external loads such as backfill. The following compressive stress limit is recommended for determining the minimum wall thickness under final prestressing combined with other external effects such as backfill.

$$\frac{P_e}{(A_g + (2n-1)A_s)} + \frac{P_h}{[A_g + (2n-1)A_s + (n-1)A_{ps}]} \leq$$

$$0.45 f'_c \text{ lb/in.}^2 \text{ (MPa)} \qquad (2\text{-}3)$$

(use f'_g if shotcrete)

2.3.3.2.5 For unusual conditions, such as those listed in Section 2.3.10, wall thickness should be determined based on analysis.

2.3.4 *Minimum concrete and shotcrete strength for walls*—Minimum concrete and shotcrete strengths f'_c and f'_g are given in Sections 3.1.4 and 3.2.4, respectively.

2.3.5 *Circumferential prestressing*

2.3.5.1 Initial stress in the prestressed reinforcement should not be more than $0.70 f_{pu}$ in wire-wrapped systems and $0.74 f_{pu}$ in strand-wrapped systems.

2.3.5.2 After deducting prestressing losses, ignoring the compressive effects of backfill, and with the tank filled to design level, there should be residual circumferential compression in the core wall. The prestressing force should result in the following minimum values:

- 200 lb/in.^{2} (1.4 MPa) throughout the entire height of wall; and
- 400 lb/in.^{2} (2.8 MPa) at the top of an open top tank, reducing linearly to not less than 200 lb/in.^{2} (1.4 MPa) at $0.6\sqrt{(rh)}$ ft [$(2.078\sqrt{(rh)}$ mm)] below the open top.

This level of residual stress is effective in limiting crack width and spacing due to temperature, moisture, and discontinuity of the shell at the top of open top tanks.

Even when the base of the wall is hinged or fixed, the prestressing force should provide the stated residual circumferential stresses, assuming the bottom of the wall is unrestrained.

2.3.5.3 The total assumed prestressing loss caused by shrinkage, creep, and relaxation should be at least 25,000 lb/in.^{2} (175 MPa).

Losses may be larger than 25,000 lb/in.^{2} (175 MPa) in tanks that are not intended for water storage or that are expected to remain empty for long periods of time (one year or longer).

When calculating prestressing loss due to elastic shortening, creep, shrinkage, and steel relaxation, consider the properties of the materials and systems used, the service environment, the load duration, and the stress levels in the concrete and prestressing steel. Refer to References 9 through 11 and ACI 209R for guidance in calculating prestressing losses.

2.3.5.4 *Spacing of prestressed reinforcement*—Minimum clear spacing between wires or strands should be 1.5 times the wire or strand diameter, or 1/4 in. (6.4 mm) for wires, and 3/8 in. (9.5 mm) for strands, whichever is greater. Maximum center-to-center spacing should be 2 in. (50 mm) for wires, and 6 in. (150 mm) for strands, except as provided for wall openings in Section 2.3.8.

2.3.5.5 *Minimum concrete cover*—Minimum cover to the prestressed reinforcement in tank walls is 1 in. (25 mm).

2.3.6 *Wall edge restraints and other secondary bending*—Wall edge restraints, discontinuities in applying prestressing, and environmental conditions result in vertical and circumferential bending. Design consideration should be given to:

- Edge restraint of deformations due to applied loads at the wall floor joint and at the wall roof joint. Various joint details have been used to minimize restraint of joint translation and rotation. These include joints that use neoprene pads and other elastomeric materials combined with flexible waterstops;
- Restraint of shrinkage and creep of concrete;
- Sequence of application of circumferential prestressing;
- Banding of prestressing for penetrations as described in Section 2.3.8;
- Temperature differences between wall and floor or roof;
- Temperature gradient through the wall; and
- Moisture gradient through the wall.

2.3.7 *Design of vertical reinforcement*

2.3.7.1 Walls in liquid-containing tanks having a steel diaphragm may be reinforced vertically with nonprestressed reinforcement.

Nonprestressed reinforcement should be proportioned to resist the full flexural tensile stress resulting from bending due to edge restraint of deformation from loads, primary prestressing forces, and other effects listed in Sections 2.3.1 and 2.3.6. The allowable stress levels in the nonprestressed reinforcement and bar spacing for limiting crack widths should be determined based on the provisions of ACI 350-01, except that the maximum allowable tensile stress in the nonprestressed reinforcement should be limited to 18,000 lb/in.^{2} (125 MPa). The cross-sectional area of the steel diaphragm can be considered as part of the required vertical nonprestressed reinforcement based upon a development length of 12 in. (300 mm).

The bending effects due to thermal and shrinkage differences between the floor and the wall or the roof, and the effects of wall thermal and moisture gradients, can be taken into account empirically in walls with a steel diaphragm by providing a minimum area of vertical reinforcement equal to 0.005 times the core wall cross section, with 1/2 of the required area placed near each of the inner and outer faces of the wall. This area is not additive to the area determined in the previous paragraph.

Alternative methods for determining the effects of thermal and moisture gradients based on analytical procedures are given in References 2, 4, 5, 12, 13, and ACI 349. An analytical method should be used when operating conditions or extremely arid regions produce unusually large thermal or moisture gradients.

2.3.7.2 Walls in liquid-containing tanks not containing a steel diaphragm should be prestressed vertically to counteract the stresses produced by bending moments caused by wall edge restraints and secondary bending (Section 2.3.6).

Vertically prestressed walls should be designed to limit the maximum flexural tensile stress after all prestressing losses to $3\sqrt{f'_c}$ lb/in.^{2} ($0.25\sqrt{f'_c}$ MPa) under the governing combination of load, wall edge restraint, secondary bending, and circumferential prestressing force. Nonprestressed reinforcement should be near the tension face. In all locations subject to tensile stresses, the area of nonprestressed reinforcement should at least equal the total flexural tensile force based on an uncracked concrete section divided by a maximum stress in the nonprestressed reinforcement of 18,000 lb/in.^{2} (125 MPa). The minimum average effective final vertical prestressing applied to the wall should be 200 lb/in.^{2} (1.4 MPa). Spacing of vertical prestressing tendons should not exceed 50 in. (1.3 m).

2.3.7.3 Walls of structures containing dry material should be designed for vertical bending using either nonprestressed or prestressed reinforcement in accordance with ACI 318-99.

2.3.7.4 Minimum cover to the nonprestressed reinforcement in tank walls is given in Section 2.1.4.3.

2.3.8 *Wall penetrations*—Penetrations can be provided in walls for manholes, piping, openings, or construction access. Care should be taken when placing prestressing wires or strands around penetrations that the minimum spacing recommendations of Section 2.3.5.4 are met.

For penetrations having a height of 2 ft (0.6 m) or less, the band of prestressed wires or strands normally required over the height of a penetration should be displaced into circumferential bands immediately above and below the penetration. Penetrations greater than 2 ft (0.6 m) in height may require specific wall designs that provide additional reinforcement at the penetrations. The total prestressing force should not be reduced as the result of a penetration.

Each band should provide approximately 1/2 of the displaced prestressing force, and the wires or strands should not be located closer than 2 in. (50 mm) to wall penetrations. The wall thickness should be adequate to support the increased circumferential compressive force adjacent to the penetration. The concrete compressive strength can be augmented by compression reinforcement adequately confined by ties in accordance with ACI 318-99 or by steel members around the opening. The wall thickness can be increased locally, adjacent to the penetration, provided that the thickness is changed gradually.

Vertical bending resulting from the banding of prestressed reinforcement should be taken into account in the wall design.

2.3.9 *Provisions for seismic-induced forces*

2.3.9.1 Tanks should be designed to resist seismic-induced forces and deformations without collapse or gross leakage. Design and details should be based upon site-specific response spectra and damping and ductility factors appropriate for the type of tank construction and seismic restraint to be used. Alternatively, when it is not feasible to obtain site specific response spectra, designs can be based upon static lateral forces that account for the effects of seismic risk, damping, construction type, seismic restraint, and ductility acceptable to the local building official.

2.3.9.2 Provisions should be made to accommodate the maximum wave oscillation (sloshing) generated by seismic acceleration. Where loss of liquid must be prevented, or where sloshing liquid can impinge on the roof, then one or both of the following provisions should be made:

- Provide a freeboard allowance; and
- Design the roof structure to resist the resulting uplift pressures.

2.3.9.3 Criteria for determining the seismic response of tanks, including sloshing of the tank contents, are given in References 14 and 15. Other methods for determining the seismic response, such as the energy method, are also given (Reference 16).

2.3.10 *Other wall considerations*—The designer should consider any unusual conditions, such as:

- Earth backfill of unequal depth around the tank;
- Concentrated loads applied through brackets;
- Internally partitioned liquid or bulk storage structures with wall loads that vary circumferentially;
- Heavy vertical loads or very large tank radii affecting wall stability;
- Storage of hot liquids;
- Wind forces on open-top tanks;
- Ice forces in environments where significant amounts of ice form inside tanks; and
- Attached appurtenances such as pipes, conduits, architectural treatments, valve boxes, manholes, and miscellaneous structures.

2.3.10.1 *Analyses for unusual design requirements*—Cylindrical shell analysis, based on the assumption of homogeneous, isotropic material behavior, should be used to evaluate unusual design requirements.

2.4—Roof design

2.4.1 *Flat concrete roofs*—Flat concrete roofs and their supporting columns and footings should be designed in accordance with ACI 318-99 and should conform to ACI 350-01.

2.4.2 *Dome roofs*

2.4.2.1 *Design method*—Concrete or shotcrete dome roofs should be designed on the basis of elastic shell analysis.

See References 4 to 7 for design aids. A circumferentially prestressed dome ring should be provided at the base of the dome shell to resist the horizontal component of the dome thrust.

2.4.2.2 *Thickness*—Dome shell thickness is governed by buckling resistance, minimum thickness for practical construction, minimum thickness to resist gas pressure, or corrosion protective cover for reinforcement.

A recommended method for determining the minimum thickness required to provide adequate buckling resistance of a monolithic concrete spherical dome shell is given in Reference 17. This method is based on the elastic theory of dome shell stability, considering the effects of creep, imperfections, and large radius-thickness ratios.

The minimum dome thickness, based on this method, is

$$h_d = r_d \sqrt{\frac{1.5 p_u}{\phi B_i B_c E_c}} \text{ in.} \tag{2-4}$$

$$\left[h_d = r_d \sqrt{\frac{1.5 \times 10^{-3} p_u}{\phi B_i B_c E_c}} \text{ mm} \right]$$

The conditions that determine the factors B_i and B_c are discussed in Reference 17. The values given for these factors in Eq. (2-6) and (2-7) are recommended for use in Eq. (2-4) when domes are designed for conditions where the minimum live load is 12 lb/ft^2 (0.57 kPa), water is stored inside the tank, the minimum dome thickness is 3 in. (75 mm), the minimum f'_c is 3000 lb/in.^{2} (21 MPa), normalweight aggregate is used, and dead load is applied (that is, shores are removed) not earlier than seven days after concrete placement following the curing requirements in ACI 301. Recommended values for the terms in Eq. (2-4) for such domes are:

p_u is the sum of dead and live loads, factored with the load factors given in ACI 318-99 for dead and live load.

$$\phi = 0.7 \tag{2-5}$$

$$B_i = \left(\frac{r_d}{r_i}\right)^2 \tag{2-6}$$

In the absence of other criteria, the maximum r_i may be taken as $1.4r_d$ (Fig. 2.5), and in this case

$$B_i = 0.5 \tag{2-7}$$

$$B_c = 0.44 + 0.003L \text{ for } 12 \text{ lb/ft}^2 < L < 30 \text{ lb/ft}^2 \tag{2-8}$$

$$[B_c = 0.44 + 0.063L \text{ for } 0.57 \text{ kPa} < L < 1.44 \text{ kPa}]$$

$$B_c = 0.53 \text{ for } L > 30 \text{ lb/ft}^2 \text{ (1.44 kPa)}$$

$$E_c = 57{,}000 \sqrt{f'_c} \text{ lb/in.}^2 \text{ for normalweight concrete} \tag{2-9}$$

$$[E_c = 4730 \sqrt{f'_c} \text{ MPa}]$$

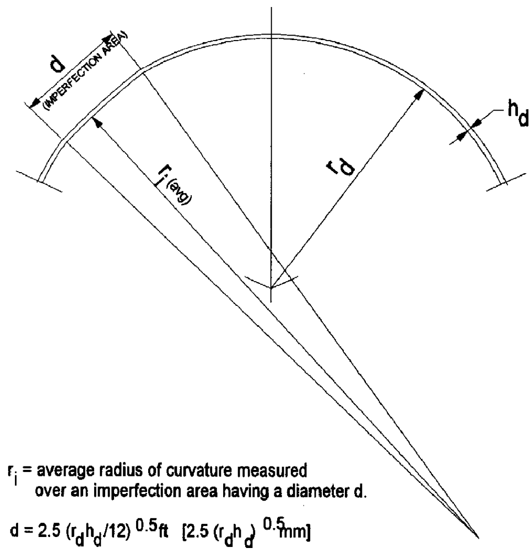

Fig. 2.5—Geometry of dome imperfection (adapted from Reference 16).

Dome shells constructed of precast concrete panels with joints between the panels that are equivalent in strength and stiffness to monolithic shells should not be thinner than the thickness obtained using Eq. (2-4).

Precast concrete panel domes with joints between panels having a strength or stiffness lower than that of a monolithic shell can be used if the minimum thickness of the panel is increased above the value given in Eq. (2-4). Such an increase should be in accordance with an analysis of the stability of the dome with a reduced stiffness as a result of joint details.

Other dome configurations, such as cast-in-place or precast domes with ribs cast monolithically with a thin shell, can be used if their design is substantiated by analysis. This analysis should show that they have buckling resistance and adequate strength to support the design live and dead loads with at least the load factors and strength reduction factors established in Reference 17 and reflected in Eq. (2-4).

Stresses and deformations resulting from handling and erection should be taken into account in the design of precast concrete panel domes. Panels should be cambered whenever the maximum dead load deflection, before incorporation as a part of the complete dome, is greater than 10% of the thickness.

The thickness of domes should not be less than 3 in. (75 mm) for monolithic concrete and shotcrete, 4 in. (100 mm) for precast concrete, and 2-1/2 in. (65 mm) for the outer shell of a ribbed dome.

2.4.2.3 *Shotcrete domes*—Dry-mix shotcrete is not recommended for domes subject to freezing-and-thawing cycles. Sand lenses caused by overspray and rebound can occur when shooting dry-mix shotcrete on relatively flat areas. These are likely to deteriorate with subsequent exposure to freezing and thawing.

2.4.2.4 *Nonprestressed reinforcement area*—For monolithic domes, the minimum ratio of nonprestressed reinforcement area to concrete area should be 0.0025 in both the parallel (circumferential) and meridional radial directions. In edge regions of thin domes and throughout domes over 5 in.

(130 mm) thick, nonprestressed reinforcement should be placed in two layers, one near each face. Minimum reinforcement should be increased for unusual temperature conditions outside normal ambient conditions.

2.4.2.5 *Dome edge region*—The edge region of the dome is subject to bending due to prestressing of the dome ring and dome live load. These bending moments should be considered in design.

2.4.2.6 *Dome ring*—The dome ring is circumferentially prestressed to counteract the horizontal component of the dome thrust.

The minimum ratio of nonprestressed reinforcement area to concrete area in the dome ring should be 0.0025 for cast-in-place dome rings. This limits shrinkage and temperature-induced crack width and spacing before prestressing.

The dome ring should be reinforced to meet the recommendations given in Section 2.1.3.2 for dead and live load factors and in Section 2.1.3.3 for strength reduction factors.

The prestressing force, after all losses, should be provided to counteract the thrust due to dead load and provide a minimum residual circumferential compressive stress to match the residual stress at the top of the wall. Additional prestressing can be provided to counteract a portion or all of the live load. If prestressing counteracts less than the full live load, additional prestressed reinforcement should be provided at reduced stress or additional nonprestressed reinforcement provided to obtain the strength recommended in Section 2.1.3.

Maximum initial stress in wires and strands should comply with Section 2.3.5.1. Maximum initial compressive stress in dome rings should comply with Section 2.3.3.2.1. Generally, a lower initial compressive stress than the maximum allowable stress is used in dome rings to limit edge bending moments in regions of the dome and wall adjacent to the dome ring.

2.4.2.7 *Minimum concrete cover*—Minimum cover to the prestressed reinforcement in the dome ring is 1 in. (25 mm).

CHAPTER 3—MATERIALS

3.1—Concrete

3.1.1 *General*—Concrete should meet the requirements of ACI 301 and ACI 350-01, except as indicated in the following.

3.1.2 *Allowable chlorides*—Maximum water-soluble chloride ions should not exceed 0.06% by mass of the cementitious material in prestressed concrete members where the concrete is not separated from the prestressed reinforcement by a steel diaphragm or in grout to avoid chloride-accelerated corrosion of steel reinforcement. Nonprestressed concrete members should meet the allowable chloride-ions limits of ACI 350-01. In prestressed concrete members where the concrete is separated from the prestressed reinforcement by a steel diaphragm, the allowable chloride-ion limits for nonprestressed concrete members may be used. ASTM C 1218 should be used to determine the level of allowable chloride ions.

3.1.3 *Exposure to freezing and thawing*—Concrete subjected to freezing-and-thawing cycles should be air-entrained in accordance with ACI 301.

3.1.4 *Compressive strength*—A minimum 28-day compressive strength of concrete should be 4000 lb/in.^{2} (28 MPa) in walls, footings, structural floors, and roofs, and 3500 lb/in.^{2} (24 MPa) in membrane floors. Walls generally experience much higher levels of compression than footings, floors, or roofs, so a higher-strength concrete in the wall can be more economical.

3.2—Shotcrete

3.2.1 *General*—Unless otherwise indicated below, shotcrete should meet the requirements of ACI 506.2 and the guidelines of ACI 506R.

3.2.2 *Allowable chlorides*—To avoid chloride-accelerated corrosion of steel reinforcement, maximum allowable chloride ions should not exceed 0.06% by mass of the cementitious material in shotcrete as determined by ASTM C 1218.

3.2.3 *Proportioning*—Shotcrete should be proportioned to the following recommendations:

- The wire coat should consist of one part portland cement and not more than three parts fine aggregate by mass; and
- The body coat should consist of one part portland cement and not more than four parts fine aggregate by mass.

3.2.4 *Compressive strength*—Minimum 28-day compressive strength of shotcrete in walls and roofs should be 4000 lb/in.^{2} (28 MPa). Shotcrete is not recommended for floors or footings.

3.2.5 *Exposure to freezing and thawing*—Dry-mix shotcrete is not recommended for domes subject to freezing-and-thawing cycles.

3.3—Admixtures

Admixtures should meet the requirements of ASTM C 494, Types A, B, C, D, or E, and be used in accordance with ACI 301. To avoid corrosion of steel in prestressed concrete, admixtures containing chloride other than from impurities in admixture ingredients should not be used. Air-entraining admixtures should comply with ASTM C 260. High-range water-reducing admixtures conforming to ASTM C 494, Type F or G, can be used to facilitate the placement of concrete.

3.4—Grout for vertical tendons

3.4.1 *General*—Vertical tendons should be post-tensioned and grouted in accordance with Section 2.1.4.2.

3.4.2 *Portland cement grout*—Grout should meet the requirements of ACI 318-99, Chapter 18. The grout, if providing expansion by the generation of gas, should have 3 to 8% total expansion measured in a 20 in. (510 mm) height starting 10 min after mixing. No visible sedimentation (bleeding) should occur during the expansion test. Grout expansion may be determined using the methods in ASTM C 940.

3.4.3 *Epoxy grout*—A moisture-insensitive epoxy grout can be used instead of a portland cement grout. Epoxy should have a low enough exotherm to ensure that it does not boil and result in a cellular structure that will not be protective to the prestressing steel. Large cavities formed by trumpets, couplers, or other tendon system hardware should be avoided when using epoxy grout to prevent heat buildup and boiling.

3.5—Reinforcement

3.5.1 *Nonprestressed reinforcement*

3.5.1.1 Nonprestressed steel reinforcing bars and welded wire fabric should be in accordance with ACI 301.

3.5.1.2 Strand for wall-to-footing seismic cables should be galvanized or protected with an epoxy coating. Galvanized strands should meet the requirements of ASTM A 416, Grade 250 or 270, before galvanizing, and ASTM A 586, ASTM A 603, or ASTM A 475 after galvanizing. Zinc coating should meet the requirements of ASTM A 475, Class A, or ASTM A 603, Class A. Epoxy-coated strands should meet the requirements of ASTM A 416, Grade 250 or 270, with a fusion-bonded epoxy-coating grit impregnated on the surface, conforming to ASTM A 882.

3.5.1.3 Sheet steel diaphragm for use in the walls of prestressed concrete tanks should be vertically ribbed with adjacent and opposing channels resembling dovetail joints (Fig. 2.2 to 2.4). The base of the ribs should be wider than the throat, providing a mechanical keyway between the inner and outer concrete or shotcrete.

Steel diaphragms should meet the requirements of ASTM A 1008 and should have a minimum thickness of 0.017 in. (0.43 mm). Some tanks use galvanized steel diaphragms. When a galvanized diaphragm is used, hot-dipped galvanized sheet steel should comply with ASTM A 653. The weight of zinc coating should not be less than G90 of Table 1 of ASTM A 653. Steel diaphragms should be continuous for the full height of the wall. Adjoining diaphragm sheets are spliced together vertically as described in Sections 3.7.1 and 4.1.3.5. Horizontal splices are not permitted.

3.5.2 *Circumferential prestressed reinforcement*

3.5.2.1 Circumferential prestressed reinforcement should be wires or strands complying with the following ASTM designations:

- *Field die-drawn wire-wrapping systems*—ASTM A 821 or with the physical and chemical requirements in ASTM A 227;
- *Other wire-wrapping systems*—ASTM A 227, ASTM A 421, or ASTM A 821; and
- *Strand-wrapping systems*—ASTM A 416.

3.5.2.2 Uncoated steel is generally used for prestressed wire reinforcement. Some wire-wrapped, and a majority of strand-wrapped, tanks have been constructed with galvanized prestressed reinforcement.

3.5.2.3 When galvanized wire or strand is used for prestressed reinforcement, the wire or strand should have a zinc coating of 0.85 oz./ft ^{2} (260 g/m^2) of uncoated wire surface, except for wire that is stressed by die drawing. If die drawing is used, the coating can be reduced to 0.50 oz./ft ^{2} (150 g/m^2) of wire surface after stressing. The coated wire or strand should meet the minimum elongation requirements of ASTM A 421 or ASTM A 416. The coating should meet the requirements for Table 4, Class A coating, specified in ASTM A 586.

3.5.2.4 Splices for prestressed reinforcement should be made of ferrous material and be able to develop the specified tensile strength of the reinforcement.

3.5.3 *Vertical prestressed reinforcement*—Vertical prestressed reinforcement should be tendons complying with one of the following ASTM specifications:

- *Strand*-ASTM A 416; and
- *High-strength steel bar*—ASTM A 722.

3.5.3.1 *Ducts*—Ducts for grouted tendons should comply with the provisions of ACI 318-99, Section 18.17. They should be watertight to prevent the entrance of cement paste from the concrete. Ducts may be rigid, semirigid, or flexible.

Rigid or semirigid ducts should be used when the tendons are placed in the ducts after the concrete is placed. Flexible ducts can be used when the tendons are installed in the ducts before concrete is placed. Ducts may be made of ferrous metal or plastic.

Duct material should not react with alkalies in the cement and should be strong enough to retain its shape and resist damage during construction. Sheathing should not cause electrolytic action or deterioration with other parts of the tendon. Semirigid ducts should be galvanized.

Plastic ducts should be watertight and directly connected to the anchorage. They should not degrade in the environment in which they will be placed and should be of adequate thickness and toughness to resist construction wear and tear without puncturing or crushing.

3.6—Waterstop, bearing pad, and filler materials

3.6.1 *Waterstops*—Waterstops should be composed of plastic or other suitable materials. Plastic waterstops of polyvinyl chloride meeting the requirements of CRD-C-572 should be used. Plastic waterstops should be ribbed and should have a minimum ultimate tensile strength of 1750 lb/in.^{2} (12 MPa), ultimate elongation of 300%, and a shore hardness of 70 to 80. Splices should be made in accordance with the manufacturers' recommendations. Polyvinyl chloride waterstops should be tested using the methods in CRD-C-572 to ensure adequacy.

3.6.2 *Bearing pads*—Bearing pads should consist of neoprene, natural rubber, polyvinyl chloride, or other materials that have demonstrated acceptable performance under conditions and applications similar to the proposed application.

3.6.2.1 Neoprene bearing pads should have a minimum ultimate tensile strength of 1500 lb/in.^{2} (10 MPa), a minimum elongation of 500% (ASTM D 412), and a maximum compressive set of 50% (ASTM D 395, Method A), with a durometer hardness of 30 to 60 (ASTM D 2240, Type A Durometer). Neoprene bearing pads should comply with ASTM D 2000, Line Call-Out M2BC410 A14 B14.

3.6.2.2 Natural rubber bearing pads should comply with ASTM D 2000, Line Call Out M4AA414A13.

3.6.2.3 Polyvinyl chloride for bearing pads should meet the requirements of CRD-C-572.

3.6.3 *Sponge fillers*—Sponge filler should be closed cell neoprene or rubber meeting the requirements of ASTM D 1056, Grade 2A1 to Grade 2A4. Minimum grade sponge filler used with cast-in-place concrete walls should be Grade 2A3.

3.7—Sealant for steel diaphragm

3.7.1 *General*—Vertical joints between sheets of the diaphragm should be sealed with a polysulfide sealant, polyurethane sealant, epoxy sealant, or with a mechanical seamer.

3.7.2 *Polysulfide sealant*—Polysulfide sealant should be a two-component elastomeric compound meeting the requirements of ASTM C 920 and should permanently bond to metal surfaces, remain flexible, and resist extrusion due to hydrostatic pressure. Air-curing sealants should not be used. Sealants used in liquid-storage tanks should be a type that is recommended for submerged service and is chemically compatible with the stored liquid. Sealant application should be in accordance with the manufacturers' recommendations. Refer to ACI 503R, Chapter 5, for surface preparation before the application of the sealant.

3.7.3 *Polyurethane sealant*—Polyurethane elastomeric sealant should meet the requirements of ASTM C 920, Class 25. It should permanently bond to metal surfaces and resist extrusion due to hydrostatic pressure. Sealant should be multicomponent Type M, Grade P (for pourable), and Grade NS (for nonsag), and should be of a type that is recommended for submerged service and is chemically compatible with the stored liquid.

3.7.4 *Epoxy sealant*—Epoxy sealant should bond to concrete, shotcrete, and steel, and should seal the vertical joints between sheets of the diaphragm. Epoxy sealant should conform to the requirements of ASTM C 881, Type III, Grade 1, and should be a 100% solids, moisture-insensitive, low-modulus epoxy system. Epoxy sealant should also be of a type that is recommended for submerged service and is chemically compatible with the stored liquid. When pumped, the epoxy should have a viscosity not exceeding 10 poises (Pa•S) at 77 °F (25 °C).

3.7.5 *Mechanical seaming*—Mechanical seams should be double-folded and watertight.

3.8—Epoxy adhesives

The bond between hardened concrete and freshly mixed concrete can be increased by properly using 100% solids, moisture-insensitive, epoxy-adhesive system meeting the requirements of ASTM C 881, Type II, Grade 2. Epoxies should be of a type that is recommended for submerged service and should be chemically compatible with the stored liquid. Refer to ACI 503R for further information on epoxy adhesives and their use in concrete construction and for recommended surface preparation before the application of the sealants.

3.9—Coatings for outer surfaces of tank walls and domes

3.9.1 *Above grade*—Coatings that seal the exterior of the tank should be breathable. A breathable or breathing-type coating is a coating that is sufficiently permeable to permit transmission of water vapor without detrimental effects to itself. Breathable coatings include rubber base, polyvinyl-chloride latex, polymeric-vinyl acrylic paints, and cementitious coatings. Coating application, including before surface preparation, should be in accordance with the manufacturers' recommendations.

3.9.2 *Below grade*—Coatings should be used to provide additional corrosion protection for the prestressing steel in aggressive environments.

3.10—Additional information on coatings

3.10.1 Refer to ACI 515.1R and ACI 350-01 for information on coatings for tanks that store aggressive materials.

CHAPTER 4—CONSTRUCTION PROCEDURES

4.1—Concrete

4.1.1 *General*—Procedures for concrete construction should be as specified in ACI 301, except as in Sections 4.1.2, 4.1.3, and 4.1.4.

4.1.2 *Floors*

4.1.2.1 Concrete in floors should be placed without cold joints and, where practical, without construction joints. Site preparation and construction should be in accordance with the recommendations of ACI 302.1R, except as modified as follows. If the entire floor cannot be cast in one operation, the size and shape of the area to be continuously cast should be selected to minimize the potential for cold joints during the placing operation, considering factors such as crew size, reliability of concrete supply, time of day, and temperature.

4.1.2.2 Floors should be cured in accordance with the requirements of ACI 308 and ACI 308.1. The water curing method using ponding is the most commonly used procedure for tank floors.

4.1.3 *Cast-in-place core walls*

4.1.3.1 Concrete should be placed in each vertical segment of the wall in a single continuous operation without cold joints or horizontal construction joints.

4.1.3.2 A 1 to 2 in. (25 to 50 mm) layer of neat cement grout should be used at the base of cast-in-place walls to help prevent voids in this critical area. The grout should have about the same water-cementitious material ratio *(w/cm)* as the concrete that is used in the wall and should have the consistency of thick paint. Concrete placed over the initial grout layer should be vibrated into that layer in such a way such that it becomes well integrated.

4.1.3.3 Measuring, mixing, and transporting concrete should be in accordance with ACI 301; forming should be in accordance with ACI 347R; placing should be in accordance with ACI 304R; consolidation should be in accordance with ACI 309R; and curing should be in accordance with ACI 308 and ACI 308.1.

4.1.3.4 Concrete that is honeycombed or does not meet the acceptance criteria of ACI 301 should be removed to sound concrete and repaired in accordance with the requirements of ACI 301.

4.1.3.5 When cast-in-place core walls are cast with a steel diaphragm, the edges of adjoining diaphragm sheets should be joined to form a watertight barrier. Mating edges should be sealed as recommended in Section 3.7.l.

4.1.4 *Precast concrete core walls*

4.1.4.1 Concrete for each precast concrete wall panel should be placed in one continuous operation without cold joints or construction joints. Panels should be cast with proper curvature.

4.1.4.2 Precast concrete wall panels should be erected to the correct vertical and circumferential alignment within the tolerances given in Section 4.6.

4.1.4.3 When precast wall panels are cast with a steel diaphragm, the edges of the diaphragm of adjoining wall panels should be joined to form a water-tight barrier. Mating edges should be sealed as recommended in Section 3.7.l.

4.1.4.4 The vertical slots between panels should be free of foreign substances. Concrete surfaces in the slots should be clean and damp before filling the slots. The slots should be filled with cast-in-place concrete, cement sand mortar, or shotcrete compatible with the joint details. The strength of the concrete, mortar, or shotcrete should not be less than that specified for concrete in the wall panels.

4.2—Shotcrete

4.2.1 *Construction procedures*—Procedures for shotcrete construction should be as specified in ACI 506.2 and as recommended in ACI 506R, except as modified as follows. The nozzle operator should be certified in accordance with ACI 506.3R.

4.2.2 *Shotcrete core walls*—Shotcrete core walls should be built up of individual layers of shotcrete, 2 in. (50 mm) or less in thickness. Thickness should be controlled as indicated in Section 4.2.5.

4.2.3 *Surface preparation of core wall*—Before applying prestressed reinforcement, dust, efflorescence, oil, and other foreign material should be removed, and defects in the core wall should be filled flush with mortar or shotcrete that is bonded to the core wall. To provide exterior surfaces to which the shotcrete can bond, concrete core walls should be cleaned by abrasive blasting or other suitable means.

4.2.4 *Shotcrete cover coat*

4.2.4.1 Externally applied circumferential prestressed reinforcement should be protected against corrosion and other damage by a shotcrete cover coat.

4.2.4.2 *The shotcrete cover coat generally consists of two or more coats*—A wire coat placed on the prestressed reinforcement and a body coat placed on the wire coat. If the shotcrete cover coat is placed in one coat, the mixture should be the same as the wire coat. Shotcrete can be applied by both manual and automated methods. When using automated methods, shotcrete is applied by nozzles mounted on power-driven machinery, a uniform distance from the wall surface, traveling at uniform speed around the wall circumference.

4.2.4.3 *Wire coat*—Each layer of circumferential prestressed wire or strand should be covered with a wire coat of shotcrete that encases the wires or strands and provides a minimum cover of 1/4 in. (6 mm) over the wire or strand. The wire coat should be applied as soon as practical after prestressing. Nozzle distance and the plasticity of the mixture are equally critical to satisfactorily encase the prestressed reinforcement. The shotcrete consistency should be plastic but not dripping.

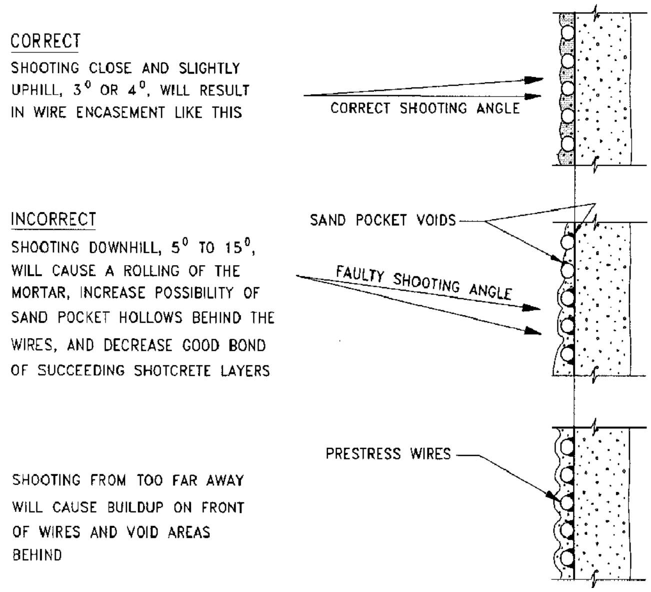

Fig. 4.1—Shotcreting prestressing steel.

The nozzle should be held at a small upward angle not exceeding 5 degrees and should be constantly moving, without shaking, and always pointing toward the center of the tank. The nozzle distance from the prestressed reinforcement should be such that the shotcrete does not build up over, or cover, the front faces of the wires or strands until the spaces between are filled. If the nozzle is held too far back, the shotcrete will deposit on the face of the wire or strand at the same time that it is building up on the core wall, thereby not filling the space behind. This condition is readily apparent and should be corrected immediately by adjusting the nozzle distance, air volume, and, if necessary, the water content.

After the wire coat is in place, visual inspection can determine whether the reinforcement was properly encased. Where the prestressed reinforcement patterns show on the surface as distinct continuous horizontal ridges, the shotcrete has not been driven behind the reinforcement and voids can be expected. If, however, the surface is substantially flat and shows virtually no pattern, a minimum of voids can be expected.

Results of correct and incorrect shotcreting techniques are illustrated in Fig. 4.1. Shotcrete placed incorrectly should be removed and replaced. The wire coat should be damp cured by a constant spray or trickling of water down the wall, except that curing can be interrupted during continuous prestressing operations. Curing compounds should not be used on surfaces that will receive additional shotcrete because they interfere with the bonding of subsequent shotcrete layers.

4.2.4.4 *Body coat*—The body coat is the final protective shotcrete layer applied on top of the outermost wire coat. The combined thickness of the outermost wire coat and the body coat should provide at least 1 in. (25 mm) of shotcrete cover over the outermost surface of the prestressed wires, strands, or embedded items (for example, clamps and splices). If the body coat is not applied as a part of the wire coat, laittance and loose particles should be removed from the surface of the wire coat before applying the body coat. Thickness control should be as recommended in Section 4.2.5. The completed shotcrete coating should be cured for at least seven days by methods specified by ACI 506.2 or until protected with a sealing coat. Curing should be started as soon as possible without damaging the shotcrete.

4.2.4.5 After the cover coat has cured, the surface should be checked for hollow-sounding or drummy spots by tapping with a light hammer or similar tool. Such spots indicate a lack of bond between coats and should be repaired. These areas should be repaired by removal and replacement with properly bonded shotcrete or by epoxy injection. If epoxy injection methods are used to repair internal voids, extreme care should be taken to ensure total filing of the void and avoid blowouts during epoxy injection.

4.2.5 *Thickness control of shotcrete core walls and cover coats*

4.2.5.1 Vertical screed wires should be installed to establish a uniform and correct thickness of shotcrete. The wires should be positioned under tension to define the outside surface of the shotcrete from top to bottom. Wires generally are 18 to 20 gage, high-strength steel wire or 150 lb (68 kg) test monofilament line spaced a maximum of 36 in. (0.9 m) apart, circumferentially.

4.2.5.2 Other methods can be used that provide positive control of the thickness.

4.2.6 *Cold-weather shotcreting*—If shotcreting is not started until the temperature is 35 °F (1.7 °C) and rising and is terminated when the temperature is 40 °F (4.4 °C) and falling, it is not considered to be cold-weather shotcreting and no provisions are needed for protecting the shotcrete against low temperatures. Shotcrete placed below these temperatures should be protected in accordance with ACI 506.2. Shotcrete should not be placed on frozen surfaces.

4.3—Forming

4.3.1 *Formwork*—Formwork should be constructed in accordance with the recommendations of ACI 347R.

4.3.2 *Slipforming*—Slipforming is not used in the construction of wire- or strand-wrapped tanks due to the potential for cold joints and honeycombs that often result in leaks.

4.3.3 *Wall form ties*—Form ties that remain in the walls of structures used to contain liquids should be designed to prevent seepage or flow of liquid along the embedded tie. Ties with snug-fitting rubber washers or O-rings are acceptable for this purpose. Tie ends should be recessed in concrete to meet the minimum cover requirements. The holes should be filled with a thoroughly bonded noncorrosive filler of strength equal to or greater than the concrete strength. Taper ties can be used instead of ties with waterstops when tapered vinyl plugs and grout are used after casting to fill the voids created by the ties.

4.3.4 *Steel diaphragms*

4.3.4.1 All vertical joints between diaphragms should be free of voids and sealed for liquid-tightness. The diaphragm form should be braced and supported to eliminate vibrations that impair the bond between the diaphragm and the concrete or shotcrete.

4.3.4.2 At the time that concrete or shotcrete is placed over the diaphragm, the steel surface can have a light coating of nonflaky oxide (rust) but be free of pitting. Diaphragms with a loose or flaky oxide coating should not be used.

4.4—Nonprestressed reinforcement

4.4.1 *Storing, handling, and placing*—Nonprestressed steel reinforcement should be fabricated, stored, handled, and placed in conformance with ACI 301.

4.4.2 *Concrete and shotcrete cover*

4.4.2.1 The minimum concrete and shotcrete cover over the steel diaphragm and nonprestressed reinforcement should be as recommended in Section 2.1.4.3. The shotcrete cover coat can be considered as part of the cover over the diaphragm.

4.4.2.2 A minimum cover of 1/2 in. (13 mm) of shotcrete should be placed over the steel diaphragm before prestressed reinforcement is placed on the core wall.

4.5—Prestressed reinforcement

4.5.1 *Wire or strand winding*

4.5.1.1 *General*—This section covers the application of high tensile strength wire or strand, wound under tension by machines around circular concrete or shotcrete walls, dome rings, or other tension-resisting structural components. Storing, handling, and placing of prestressed reinforcement should meet the requirements of ACI 301. Prestressed reinforcement should be stored and protected from corrosion.

4.5.1.2 *Qualifications*—The winding system used should be capable of consistently producing the specified stress at every point around the wall within a tolerance of ±7% of the specified initial stress in each wire or strand (Section 2.3.5.1).

Winding should be under the direction of a supervisor with technical knowledge of prestressing principles and experience with the winding system being used.

4.5.1.3 *Anchoring of wire or strand*—To minimize the loss of prestressing in case of a break during wrapping, each coil of prestressed wire or strand should be anchored to an adjacent wire or strand or to the wall surface at regular intervals. Anchoring clamps should be removed when the clamp cover in the completed structure is less than 1 in. (25 mm).

4.5.1.4 *Splicing of wire or strand*—Ends of the individual coils should be joined by ferrous splicing devices as recommended in Section 3.5.2.4.

4.5.1.5 *Concrete or shotcrete strength*—Concrete or shotcrete compressive strength at the time of stressing should be at least 1.8 times the maximum initial compressive stress due to prestressing in any wall section. The compressive strength is evaluated by testing representative samples in accordance with ACI 318-99 for concrete and ACI 506.2 for shotcrete.

4.5.1.6 *Stress measurements and wire or strand winding records*—A calibrated stress-recording device that can be readily recalibrated should be used to determine stress levels in prestressed reinforcement throughout the wrapping process. At least one stress reading for every coil of wire or strand, or for each 1000 lb (455 kg), or for every 1 ft (0.3 m) of height of wall per layer, should be taken after the prestressed reinforcement has been applied on the wall. Readings should be made when the wire or strand has reached ambient temperature. All such readings should be made on straight lengths of prestressed reinforcement. A written record of stress readings, including location and layer, should be maintained. This submission should be reviewed by the engineer or another representative of the owner before accepting the work. Continuous electronic recordings taken on the wire or strand in a straight line between the stressing head and the wall can be used instead of periodic readings when the system allows no loss of tension between the reading and final placement on the wall. The total initial prestressing force measured on the wall per-unit-height should not be less than the specified initial force in the locations indicated on the design force diagram and not more than 5% greater than the specified force.

4.5.1.7 *Prestressed reinforcement stress adjustment*—If the stress in the installed reinforcement is less than specified, additional wire or strand should be applied to correct the deficiency. If the stress exceeds 107% of specified, the wrapping operation should be discontinued immediately, and adjustments should be made before wrapping is restarted. Broken prestressing wires or strands should be removed from the previous clamp or anchorage and should not be reused.

4.5.1.8 *Spacing of prestressed reinforcement*—Spacing of wire and strand should be as recommended in Section 2.3.5.4. Wire or strand in areas adjacent to wall penetrations or inserts should be uniformly spaced as described in Section 2.3.8.

4.5.2 *Vertical prestressing tendons*

4.5.2.1 *General*—Tendons should meet applicable construction requirements specified in ACI 301 and design provisions required in ACI 318-99, unless modified in this section.

4.5.2.2 *Qualifications of supervisor*—Field handling of tendons and associated stressing and grouting equipment should be under the direction of a supervisor who has technical knowledge of prestressing principles and experience with the particular system of post-tensioning being used.

4.5.2.3 *Duct installation*—Ducts for internally grouted vertical prestressing tendons should be secured to prevent distortion, movement, or damage from placement and vibration of the concrete. Ducts should be supported to limit wobble. After installing the forms, the ends of the ducts should be covered to prevent entry of mortar, water, or debris. Ducts should be inspected before concreting, looking for holes that would allow mortar leakage or indentations that restrict movement of the prestressed reinforcement during the stressing operation. If prestressed reinforcement is installed in the ducts after the concrete has been placed, the ducts should be free of mortar, water, and debris immediately before installing the prestressed steel. When ducts are subject to freezing before grouting, they should be protected from entry of rain, snow, or ice, and drainage should be provided at the low point to prevent damage from freezing water.

4.5.2.4 *Installation and tensioning of vertical tendons*—Vertical prestressing tendons should be tensioned by hydraulic jacks. The effective force in the prestressed reinforcement should not be less than that specified. The jacking force and elongation of each tendon should be recorded and reviewed by the engineer or another representative of the owner before accepting the work.

The prestressed reinforcement should be free and unbonded in the duct before post-tensioning. Concrete compressive strength at the time of stressing should be at least 1.8 times the maximum initial compressive stress acting on any net wall cross section and sufficient to resist the concentration of bearing stress under the anchorage plates without damage.

Total or partial prestressing should be applied before wrapping. Vertical prestressing should be done in the sequence specified. This sequence should be detailed on the post-tensioning shop drawings.

Forces determined from tendon elongation measurements and from the observed jacking pressure should be in accordance with ACI 318-99.

4.5.2.5 *Grouting*—All vertical tendons should be protected from corrosion by completely filling all voids in the tendon system with hydraulic cement grout or epoxy grout.

Grouting should be carried out as promptly as possible after tensioning. The total exposure time of the prestressing tendon (other than in a controlled environment) before grouting should not exceed 30 days, nor should the period between tensioning and grouting exceed seven days, unless precautions are taken to protect the prestressing tendon against corrosion. The methods or products used should not jeopardize the effectiveness of the grout to protect against corrosion nor the bond between the prestressed tendon and the grout. For potentially corrosive environments, additional restrictions can be required. Grouting equipment should be capable of grouting at a pressure of at least 200 lb/in.^{2} (1.4 MPa).

The prestressed steel in each vertical tendon should be fully encapsulated in grout. Grout injection should be from the lowest point in the tendon to avoid entrapping air.

All grout should pass through a maximum No. 200 (4.75 μm) sieve before going into the grout pump. When hot-weather conditions contribute to quick setting of grout, the grout should be cooled to prevent blockages during pumping operations by methods such as cooling the mixing water. When freezing-weather conditions prevail during and following the placement of grout, the grout should be protected from freezing until it attains a strength of 500 lb/in.^{2} (3.5 MPa) (ACI 306R).

4.5.2.6 *Protecting vertical post-tensioning anchorages*—Recessed end-anchorages should be protected in accordance with ACI 318-99, Chapter 18.

4.6—Tolerances

4.6.1 *Tank radius*—The maximum permissible deviation from the specified tank radius should be 0.1% of the radius or 60% of the core wall thickness, whichever is less, and should be measured to the inside wall face.

4.6.2 *Localized tank radius*—The maximum permissible deviation of the tank radius along any 10% of circumference,

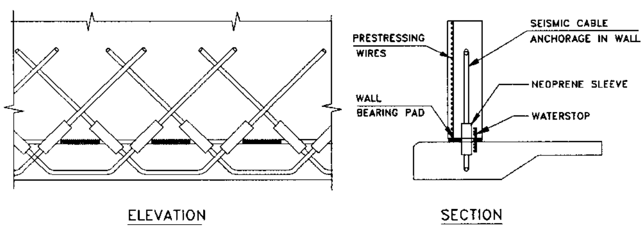

Fig. 4.2—Seismic cable details.

measured to the inside wall face, should be 5% of the core wall thickness.

4.6.3 *Vertical walls*—Walls should be plumb within 3/8 in. per 10 ft (9.5 mm per 3.0 m) of vertical dimension.

4.6.4 *Wall thickness*—Wall thickness should not vary more than –1/4 in. (–6.4 mm) or +1/2 in. (+13 mm) from the specified thickness.

4.6.5 *Precast panels*—The mid-depths of adjoining precast concrete panels should not vary inwardly or outwardly from one another in a radial direction by more than 3/8 in. (9.5 mm).

4.6.6 *Concrete domes*—The average radius of curvature of any dome surface imperfection (Fig. 2.5) with a minimum in-plane diameter of $2.5\sqrt{r_d h_d / 12}$ ft $(2.5\sqrt{r_d h_d}$ mm) should not exceed $1.4r_d$ (Reference 17).

4.7—Seismic cables

When seismic cables (Fig. 4.2) are installed in floor-wall or roof-wall connections to provide tangential (membrane) resistance to lateral motion between the wall and the footing or roof, the following details should be noted:

4.7.1 *Separation sleeves*—Sleeves of rubber or other elastomeric material should surround the seismic cables at the joint to permit radial wall movements independent from the cables. Concrete or grout should be prevented from entering the sleeves.

4.7.2 *Protection*—The cable should be galvanized or protected with a fusion-bonded epoxy coating, grit-impregnated on the surface. The portion of the cable not enclosed by sleeves should be anchored to the wall concrete or shotcrete and to the footing or roof concrete.

4.7.3 *Placing*—Cables should be cut to uniform lengths before being placed in the forms. Care should be taken during placement to avoid compression of the bearing pad and consequent restraint of radial wall movement.

4.8—Waterstops

Waterstops should be secured to ensure positive positioning by split forms or other means. Waterstops that are not accessible during concreting should be tied to reinforcement or otherwise fixed to prevent displacement during concrete placement operations.

A horizontal waterstop that is accessible during concreting should be secured in a manner allowing it to be bent up while the concrete is placed and compacted underneath, after which it should be allowed to return to position, and the additional concrete placed over the waterstop.

All waterstops should be spliced in a manner that ensures continuity as a water barrier.

4.9—Elastomeric bearing pads

4.9.1 *Positioning*—Bearing pads should be attached to the concrete with a moisture-insensitive adhesive or other positive means to prevent uplift during concreting. Pads in cast-in-place concrete walls should be protected from damage from nonprestressed reinforcement by inserting small, dense concrete blocks on top of the pad, directly under the nonprestressed reinforcement ends. The pads should not be nailed, unless they are specifically detailed for such anchorage.

4.9.2 *Free-sliding joints*—When the wall-floor joint is designed to translate radially, the joint should be detailed and constructed to ensure free movement of the wall base.

4.10—Sponge-rubber fillers

4.10.1 *General*—Sponge-rubber fillers at wall-floor joints should be of sufficient width and correctly placed to prevent voids between the sponge rubber, bearing pads, and waterstops. Fillers should be detailed and installed to provide complete separation between the wall and the floor. The method of securing sponge-rubber pads is the same as for elastomeric bearing pads.

4.10.2 *Voids*—All voids and cavities occurring between butted ends of bearing pads, between pads and waterstops, and between pads and joint fillers should be filled with nontoxic sealant compatible with the materials of the pad, filler, waterstop, and the submerged surface. Concrete-to-concrete hard spots that would inhibit free translation of the wall should not exist.

4.11—Cleaning and disinfection

4.11.1 *Cleaning*—After tank construction has been completed, all trash, loose material, and other items of a temporary nature should be removed from the tank. The tank should be thoroughly cleaned with a high-pressure water jet, sweeping, scrubbing, or other means. All water and dirt or foreign material accumulated in this cleaning operation should be discharged from the tank or otherwise removed. All interior surfaces of the tank should be kept clean until final acceptance. After cleaning is completed, the vent screen, overflow screen, and any other screened openings should be in satisfactory condition to prevent birds, insects, and other possible contaminants from entering the facility.

4.11.2 *Disinfection*

4.11.2.1 Potable water storage tanks should be disinfected in accordance with AWWA C 652.

CHAPTER 5—ACCEPTANCE CRITERIA FOR LIQUID-TIGHTNESS OF TANKS

5.1—Test recommendations

5.1.1 *Liquid-tightness test*—When the tank is designed to contain liquid, a test for liquid-tightness should be performed by the contractor and observed by the engineer or another representative of the owner. The test should measure the leakage with a full tank over a period of at least 24 h by measuring the drop in water level, taking into consideration loss from evaporation. Guidance for the determination of evaporation loss is provided in ACI 350.1-01.

Alternatively, the following test for liquid-tightness can be used. The tank is maintained full for three days before beginning the test. The drop in liquid level should be measured over the next five days to determine average daily leakage for comparison with the allowable leakage given in Section 5.2.

5.2—Liquid-loss limit

5.2.1 *Maximum limit*—Liquid loss in a 24 h period should not exceed 0.05% of the tank volume.

5.2.2 *Special conditions*—Where the supporting soils are susceptible to piping action or swelling, or where loss of the contents would have an adverse environmental impact, more stringent liquid-loss limits may be more appropriate than those recommended in Section 5.2.1.

5.2.3 *Inspection*—If liquid loss in a 24 h period exceeds 0.025% of the tank volume, the tank should be inspected for point sources of leakage. If point sources are found, they should be repaired.

5.3—Visual criteria

5.3.1 *Moisture on the wall*—Damp spots on the exterior wall surface are unacceptable. Damp spots are defined as spots where moisture can be picked up on a dry hand.

5.3.2 *Floor-wall joint*—Leakage that includes visible flow through the wall-floor joint is unacceptable. Dampness on top of the footing should not be construed as flowing water.

5.3.3 *Groundwater*—Floors, walls, and wall-floor joints should not allow groundwater to leak into the tank.

5.4—Repairs and retesting

Repairs should be made if the tank fails the liquid-tightness test, the visual criteria, or is otherwise defective. After repair, the tank should be retested to confirm that it meets the liquid-tightness criteria and visual criteria.

CHAPTER 6—REFERENCES

6.1—Referenced standards and reports

The standards and reports listed as follows were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version, except where a specific year designation is given.

American Concrete Institute

116R Cement and Concrete Terminology
207.1R Mass Concrete
207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete
209R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
301 Specifications for Structural Concrete
302.1R Guide for Concrete Floor and Slab Construction
304R Guide for Measuring, Mixing, Transporting, and Placing Concrete
306R Cold Weather Concreting
308 Standard Practice for Curing Concrete
308.1 Standard Specification for Curing Concrete
309R Guide for Consolidation of Concrete
313 Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials and Commentary
318-99 Building Code Requirements for Structural Concrete and Commentary
347R Guide to Formwork for Concrete
349 Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary
350-01 Code Requirements for Environmental Engineering Concrete Structures and Commentary
350.1-01 Tightness Testing of Environmental Engineering Concrete Structures
503R Use of Epoxy Compounds with Concrete
506R Guide to Shotcrete
506.2 Specification for Shotcrete
506.3 Guide to Certification of Shotcrete Nozzlemen
515.1R A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete

ASTM International

A 227M Specification for Steel Wire, Cold-Drawn for Mechanical Springs
A 416/ A 416M Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A 421/ A 421M Specifications for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A 475 Specification for Zinc-Coated Steel Wire Strand
A 586 Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand and Zinc-Coated Wire for Spun-In-Place Structural Strand
A 603 Specification for Zinc-Coated Steel Structural Wire Rope
A 653/ A 653M Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannnealed) by the Hot-Dip Process
A 722 A 722M Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete
A 821/ A 821M Specification for Steel Wire, Hard-Drawn for Prestressing Concrete Tanks
A 882/ A 822M Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand
A 1008 A 1008M Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability
C 260 Specification for Air-Entraining Admixtures for Concrete
C 494 C 494M Specification for Chemical Admixtures for Concrete
C 881 Specification for Epoxy-Resin-Base Bonding Systems for Concrete
C 920 Specification for Elastomeric Joint Sealants
C 940 Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory
C 1218\ C 1218M Test Method for Water-Soluble Chloride in Mortar and Concrete
D 395 Test Methods for Rubber Property-Compression Set

D 412 Testing Methods for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers-Tension
D 1056 Specification for Flexible Cellular Materials-Sponge or Expanded Rubber
D 2000 Classification System for Rubber Products in Automotive Applications
D 2240 Test Method for Rubber Property-Durometer Hardness

American Water Works Association
C 652 Disinfection of Water-Storage Facilities

U.S. Army Corps of Engineers Specifications
CRD-C-572 Specification for PVC Waterstop

The above publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, Mich. 48333-9094

ASTM International
100 Barr Harbor Dr.
West Conshohocken, Pa. 19428-2959

American Water Works Association
6666 West Quincy Ave.
Denver, Colo. 80235

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Rd.
Vicksburg, Miss. 39180

6.2—Cited references

1. Vitharana, N. D., and Priestley, M. J. N., "Significance of Temperature-Induced Loadings on Concrete Cylindrical Reservoir Walls," *ACI Structural Journal*, V. 96, No. 5, Sept.-Oct. 1999, pp. 737-747.

2. Timoshenko, S., and Woinowsky-Krieger, S., *Theory of Plates and Shells*, 2nd Edition, McGraw-Hill, New York, 1959.

3. Flugge, W., *Stresses in Shells*, Springer-Verlag, New York, 1967.

4. Baker, E. H.; Kovalevsky, L.; and Rish, F. L., *Structural Analysis of Shells*, McGraw-Hill, New York, 1972.

5. Ghali, A., *Circular Storage Tanks and Silos*, E&FN Spon, Ltd., London, 1979, 352 pp.

6. Billington, D. P., *Thin Shell Concrete Structures*, 2nd Edition, McGraw-Hill, New York, 1982, 373 pp.

7. Heger, F. J.; Chambers, R. E.; and Dietz, A. G., "Thin Rings and Shells," *Structural Plastics Design Manual*, American Society of Civil Engineers, New York, 1984, pp. 9-1 to 9-145.

8. Ghali, A., and Favre, R., *Concrete Structures; Stresses and Deformations*, Chapman and Hall, London and New York, 1986.

9. Magura, D. D.; Sozen, M. A.; and Siess, C. P., "A Study of Stress Relaxation in Prestressing Reinforcement," *Journal of the Prestressed Concrete Institute*, V. 9, No. 2, 1964, pp. 13-57.

10. PCI Committee on Prestress Losses, "Recommendations for Estimating Prestress Losses," *Journal of the Prestressed Concrete Institute*, V. 20, No. 4, 1975, pp. 43-75.

11. Zia, P.; Preston, H.; Kent S.; Norman L.; and Workman, E. B., "Estimating Prestress Losses," *Concrete International*, V. 1, No. 6, June 1979, pp. 32-38.

12. Priestley, M. J. N., "Ambient Thermal Stresses in Circular Prestressed Concrete Tanks," ACI JOURNAL, *Proceedings* V. 73, No. 10, Oct. 1976, pp. 553-560.

13. Hoffman, P. C.; McClure, R. M; and West, H. H., "Temperature Study of an Experimental Concrete Segmental Bridge," *Journal of the Prestressed Concrete Institute*, V. 28, No. 2, 1983, pp. 78-97.

14. United States Nuclear Regulatory Commission (formerly United States Atomic Energy Commission), Division of Technical Information, TID-7024, *Nuclear Reactors and Earthquakes*, National Technical Information Service, 1963.

15. American Water Works Association, ANSI/AWWA D110-95, "Standard for Wire-Wound Circular Prestressed Concrete Water Tanks," American Water Works Association, Denver, Colo., 1996.

16. Housner, G. W., "Limit Design of Structures to Resist Earthquakes," *Proceedings of the World Conference on Earthquake Engineering*, Berkeley, Calif., 1956.

17. Zarghamee, M. S., and Heger, F. J., "Buckling of Thin Concrete Domes," ACI JOURNAL, *Proceedings* V. 80, No. 6, Nov.-Dec. 1983, pp. 487-500.

APPENDIX A—RECOMMENDATIONS AND CONSIDERATIONS RELATED TO THE DESIGN AND CONSTRUCTION OF TANK FOUNDATIONS

A.1—Scope

This appendix presents information related to the design and construction of foundations for circular-wrapped, prestressed concrete tanks.

A.2—Subsurface investigation

A.2.1 The subsurface conditions at a site should be known to determine the soil-bearing capacity, compressibility, shear strength, and drainage characteristics. This information is generally obtained from soil borings, test pits, load tests, sampling, laboratory testing, and analysis by a geotechnical engineer.

A.2.2 Once the location and diameter of the proposed tank is determined, boring locations can be established at the site. The ground surface elevations at each of the boring locations should be obtained.

The following boring layout is recommended: one boring at the center of the tank, plus a series of equally spaced borings around the plan footprint of the tank wall. The distance between such borings should not exceed 100 ft (30 m) (Fig. A.1). If the tank diameter is greater than 200 ft (60 m), another four borings, equally spaced, may be taken around the perimeter of a circle whose center is the center of the plan footprint of the tank, and whose radius is 1/2 that of the tank (Fig. A.2).

Additional borings should be considered if the following conditions exist:

- Site topography is uneven;

- Fill areas are anticipated or revealed by geotechnical investigation;
- Soil strata vary horizontally rather than vertically; and
- Earthen mounds are to be placed adjacent to the tank.

A.2.3 Borings are typically taken to below the depth of significant foundation influence or to a competent stratum. At least one boring should penetrate to a depth of 75% of the tank radius or a minimum of 60 ft (18 m). All other borings should penetrate to at least a depth of 25% of the tank radius or a minimum of 30 ft (9 m). If borings encounter bedrock exhibiting variations or low-quality characteristics in the rock structure, rock corings should be made into the rock layer to provide information on the rocks' soundness. Given the wide variety of subsurface conditions that may be encountered, the geotechnical engineer should make the final determination of the appropriate number, location, and depth of the borings.

The groundwater level in the borings should be measured and recorded during the drilling, immediately after completion, and 24 h after completion.

A.2.4 Soil samples of each strata penetrated, and a measurement of the resistance of the soil to penetration, should be obtained from borings performed at the site in conformance with ASTM D 1586. Relatively undisturbed soil samples can be obtained at representative depths in conformance with ASTM D 1587. Other sampling methods are used where appropriate. Recovered soil samples should be visually classified and tested in the laboratory for the following:

- Natural moisture content (ASTM D 2216);
- Particle-size distribution (ASTM D 422);
- Atterberg limits (ASTM D 4318);
- Shearing strength (ASTM D 2166); and
- Compressibility of the soil (ASTM D 2435).

Additional testing should be performed when necessary to obtain a sufficient understanding of the underlying soil characteristics at the site.

A.3—Design considerations

A.3.1 The allowable bearing capacity for normal operating conditions (static loading) should be determined by dividing the ultimate capacity by a factor of three. This factor of safety can be reduced to 2.25 when combining operating loading conditions with transient loading conditions, such as wind or earthquake.

A.3.2. Typical modes of settlement for shallow tank foundations and their recommended maximum limits are:

- Uniform settlement of the entire tank foundation should be limited to a maximum of 6 in. (150 mm);
- Uniform (planar) tilting (when the tank foundation tilts uniformly to one side) should be limited to a maximum of 3/8 in. drop per 10 ft (9.5 mm per 3.0 m);
- Angular distortion should be limited to 1/4 in. drop per 10 ft (6 mm per 3.0 m) of the foundation diameter; and
- A maximum combined uniform and tilting settlement at the tank foundation perimeter should be limited to 6 in. (150 mm), unless hydraulic requirements dictate a lesser value.

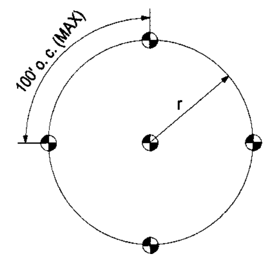

Fig. A.1—Recommended boring layout for tank diameters ≤ 200 ft (60 m). Note: For tank diameters less than 50 ft, the number of perimeter borings may be reduced.

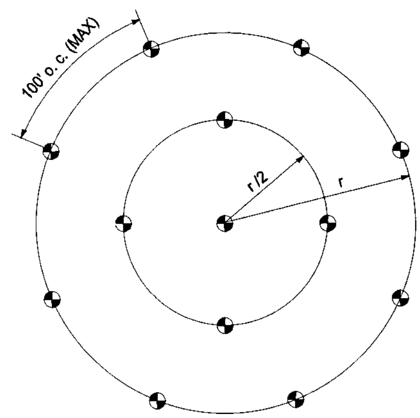

Fig. A.2—Recommended boring layout for tank diameters > 200 ft (60 m).

Exterior piping connections to the tank should be designed to tolerate the anticipated settlements. A conical (dish-shaped) settlement is the classic settlement mode for a cylindrical tank foundation founded upon uniform soil conditions. A conservative estimate of the maximum tolerable differential settlement between the tank center and the perimeter of a uniform thickness circular membrane floor slab can be expressed by the equation

$$y = 3 \times 10^{-3} \times \frac{r^2}{t} \text{ in. } \left[y = 21 \times 10^{-6} \times \frac{r^2}{t} \text{ mm} \right] \quad \text{(A3-1)}$$

where

y = maximum tolerable differential settlement between the tank perimeter and tank center, in. (mm);

r = tank radius, ft (mm); and

t = floor slab thickness, in. (mm).

Localized settlement (subsidence) beneath the tank foundation can occur as a result of localized areas of supporting soils that exhibit higher degrees of settlement than other areas of the supporting soil. Conversely, if supporting soils contain unyielding hard spots, such as a boulder or bedrock pinnacle, a higher degree of settlement is typically experienced in the supporting soils surrounding the hard spots. Soil types that exhibit shrinkage and swell potential can have similar effects. If undesirable areas of soil, rock, or both are discovered during field-testing or construction, it is advisable to remove and replace with a suitable compacted material to a depth recommended by a geotechnical engineer.

A.3.3 Backfill is usually placed at elevations that will be compatible with the surrounding site grading. Backfill should be placed around the tank to a sufficient depth to provide frost protection for the tank perimeter footing. Backfill material should be free of organic material, construction debris, and large rocks. The backfill should be placed in uniform layers and compacted. The excavated material from the tank foundation is often used as tank backfill material when suitable.

A.3.4 The site-finish grading adjacent to the tank should be sloped away from the tank wall not less than one vertical in 12 horizontal (1:12) for a horizontal distance of at least 8 ft (2.4 m). The surrounding site-finish grading should be established so that surface water runoff may be collected at areas of the site where it may dissipate into the earth or be captured into a drainage structure. Erosion protection should be provided where surface water runoff may erode backfill or foundation soil materials.

A.3.5 Site conditions that require engineering considerations are:

- Hillsides where part of a tank foundation can be on undisturbed soil or rock and part may be on fill, resulting in a nonuniform soil support;
- Adjacent to water courses or deep excavations where the lateral stability of the ground is questionable;
- Adjacent heavy structures that distribute a portion of their load to the subsoil beneath the tank site;
- Swampy or filled ground where layers of muck or compressible organics are at or below the surface, or where unstable materials may have been deposited as fill;
- Underlying soils, such as layers of plastic clay or organic clays, that can support heavy loads temporarily but settle excessively over long periods of time;
- Underlying soils with shrinkage and swell characteristics;
- Potentially corrosive site soils, such as those that are very acidic or alkaline, or those with high concentrations of sulfates or chlorides;
- Regions of high seismicity with soils susceptible to liquification; and
- Exposure to flooding or high groundwater levels.

A.3.6. If the existing subgrade is not capable of sustaining the anticipated tank foundation loading without excessive settlement, any one or a combination of the following methods may improve the condition:

- Remove the unsuitable material and replace it with a suitable compacted material;
- Preconsolidate the soft material by surcharging the area with an overburden of soil. Strip or wick drains can be used in conjunction with this method to accelerate settlement;
- Incorporate geosynthetic reinforcing materials within the foundation soils;
- Stabilize the soft material by lime stabilization, chemical methods, or injecting cement grout;
- Improve the existing soil properties using vibro-compaction, vibro-replacement, or deep dynamic compaction;
- Construct a mat foundation that distributes the load over a sufficient area of the subgrade; and
- Use a deep foundation system to transfer the load to a stable stratum beneath the subgrade. This method consists of constructing a structural concrete base slab supported by piles or piers.

A.4—Geotechnical report content

A.4.1 After completing the subsurface investigation, a detailed report should be prepared by a geotechnical engineer.

This report should include:

- The scope of the investigation;
- A description of the proposed tank, including major dimensions, elevations (including finished floor elevation), and loadings;
- A description of the tank site, including existing structures, drainage conditions, vegetation, and any other relevant features;
- Geological setting of the site;
- Details of the field exploration, such as number of borings, location of borings, and depth of borings;
- A general description of the subsoil conditions as determined from the recovered soil samples, laboratory tests, and standard penetration resistance; and
- The expected groundwater level at the site during construction and after project completion.

The geotechnical recommendations should include:

- Type of foundation system;
- Subgrade preparation, including proof-rolling and compaction; when necessary, consider the possibility of pumping during compaction of the subgrade;
- Foundation base material and placement procedure, including compaction requirements;
- Backfill material and placement procedure, where required;
- Allowable-bearing capacity;
- Estimated settlements;
- Lateral equivalent soil pressure, including active, at rest, passive, and seismic, where applicable;
- Seismic soil profile type;
- Anticipated groundwater control measures needed at the site during and after construction, including the possibility of buoyancy of the empty tank; and
- Conclusions and limitations of the investigation.

The report should have the following attachments:

- Site location map;
- A plan indicating the location of the borings with respect to the proposed tank and any existing structures on the site;
- Boring logs; and
- Laboratory test results, including Atterberg limits, unconfined compressive strength, and shear strength, where applicable.

A.5—Shallow foundation

A.5.1 When the geotechnical investigation of the subsurface soil conditions at the site indicate that the subgrade has adequate bearing capacity to support the tank loadings without exceeding tolerable settlement limits, a shallow foundation system, such as a membrane floor with a perimeter wall footing, should be used (Fig. A.3).

A.5.2 The subgrade should be of a uniform density and compressibility to minimize the differential settlement of the floor and footing. Disturbed areas of the exposed subgrade, or loosely consolidated soil, should be compacted. Areas of the subgrade that exhibit signs of soft or unstable conditions should be compacted or replaced with a suitable compacted material. When subgrade material is replaced, this material should be compacted to 95% of the maximum laboratory density determined by ASTM D 1557. The field tests for measurement of in-place density should be in conformance with ASTM D 1556 or D 2922. Particular care should be exercised to compact the soil to the specified density under and around underfloor pipe encasements. Controlled low-strength material (CLSM) is sometimes used to fill overexcavated areas or to provide a smooth working base over an irregular or unstable rock surface.

A.5.3 Base material should be placed over the subgrade when the subgrade materials do not allow free drainage. The base material should consist of a clean, well-compacted, angular or subangular granular material with a minimum thickness of 6 in. (150 mm). The gradation of the base material should be selected to permit free drainage without the loss of fines or intermixing with the subgrade material. This objective is typically achieved by limiting the amount of material that passes the No. 200 (75 μm) sieve to a maximum 8% by weight of the total base material. If a suitable base material is unavailable, a geotextile fabric should be placed between the subgrade and base material. Base material should be compacted to 95% of the maximum laboratory density determined by ASTM D 1557. The field tests for measurement of in-place density should be in conformance with ASTM D 1556 or D 2922. If the base material is cohesionless, the relative density should be measured in accordance with ASTM D 4253 and D 4254. A relative density of 70 to 75% is normally desirable. Alternatively, if the base layer is relatively thin (8 in. [200 mm] or less), ASTM compaction and density tests can be replaced by compaction performance criteria in which the maximum lift, number of passes in each direction, type, and weight of equipment are specified. Surface elevation of the base material should be +0 and –1/2 in. (+0 and –13 mm) over the entire floor area. All transitions in elevations should be smooth and gradual, varying no more than 1/4 in. per 10 ft (6 mm per 3.0 m).

A.5.4 When groundwater conditions at the tank site indicate the possibility of hydrostatic uplift occurring on the tank floor, a drainage system should be considered to prevent groundwater from rising to an undesirable level. The drainage system can discharge to a manhole or other drainage structure where the flow can be observed. The drainage structure should be located at a lower elevation than the floor slab to prevent surcharge and backflow to the tank foundation. If a drainage system is not practical, then soil anchors, tension piles, or a heavy floor can be used.

A.5.5 Backfilling may begin after the tank has been completed and tested for watertightness. Excavated material can be used for backfilling if suitable. Backfill material should be placed in uniform lifts about the periphery of the tank.

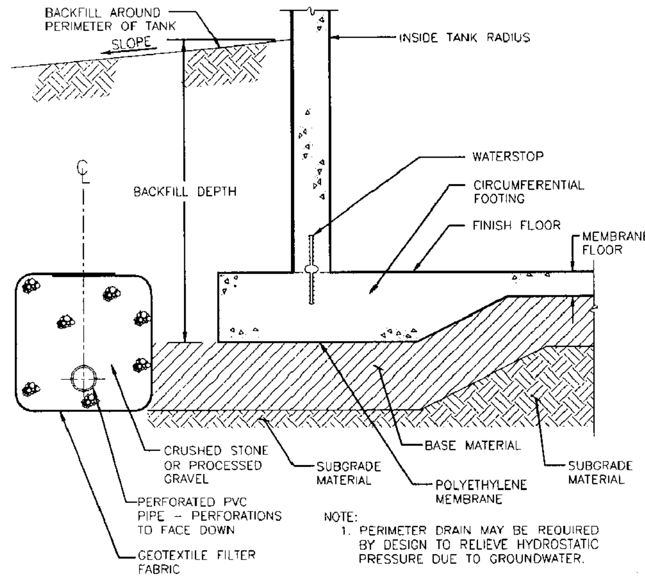

Fig. A.3—Typical tank foundation.

Each lift should be compacted to at least 90% of the maximum laboratory density determined by ASTM D 1557. The field tests for measurement of in-place density should be in conformance with ASTM D 1556 and ASTM D 2922. If the backfill material is impervious (for example, clay), it may be necessary to install a drainage blanket, such as a layer of gravel or a geotextile mesh, against the wall.

A.6—References

A.6.1 *Referenced standards and reports*

ASTM International

D 422 Test Method for Particle-Size Analysis of Soils

D 1556 Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method

D 1557 Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort

D 1586 Test Method for Penetration Test and Split-Barrel Sampling of Soils

D 1587 Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes

D 2166 Test Method for Unconfined Compressive Strength of Cohesive Soil

D 2216 Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

D 2435 Test Method for One-Dimensional Consolidation Properties of Soils

D 2922 Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)

D 4253 Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table

D 4254 Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density

D 4318 Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

ANSI/AWWA D110
Wire and Strand-Wound, Circular, Prestressed Concrete Water Tanks

A.6.2 *Other references*

American Petroleum Institute, 1997, *API Standard 650, Appendix B.*

Bowles, J., 1982, *Foundation Analysis and Design*, 3rd Edition, McGraw-Hill, New York.

Building Officials and Code Administrators International, Inc., 1996, *"The BOCA National Building Code."*

Das, B., 1990, *Principles of Foundation Engineering*, 2nd Edition, PWS-Kent Publishing Co., Boston, Mass.