Design and Construction of Circular Prestressed Concrete Structures with Circumferential Tendons

Reported by ACI Committee 373

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FOREWORD

This report provides recommendations for the design and construction of circular prestressed concrete structures (commonly referred to as “tanks”) post-tensioned with circumferential tendons. These thin cylindrical shells of either cast-in-place or precast concrete are commonly used for liquid and bulk storage. Vertical post-tensioning is often incorporated in the walls as part of the vertical reinforcement. Recommendations are applicable to circumferential prestressing achieved by post-tensioning tendons placed within the wall or on the exterior surface of the wall. Procedures to prevent corrosion of the prestressing elements are emphasized. The design and construction of dome roofs are also covered.

KEYWORDS: circumferential prestressing; concrete; corrosion resistance; domes; floors; footings; joints; loads (forces); prestressed concrete; prestressed reinforcement; reinforcing steel; roofs; shotcrete; shrinkage; tanks; temperature; tendons; walls.

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CHAPTER 1—GENERAL

1.1—Introduction
The design and construction of circular prestressed concrete structures using tendons requires specialized engineering knowledge and experience. This report reflects over four decades of experience in designing and constructing circular prestressed concrete structures with tendons. When designed and constructed by knowledgeable individuals, these structures can be expected to serve for fifty years or more without requiring significant maintenance.

This report is not intended to prevent development or use of new advances in the design and construction of circular prestressed concrete structures. This report is not intended for application to nuclear reactor pressure vessels or cryogenic containment structures.

This report describes current design and construction practices for tanks prestressed with circumferential post-tensioned tendons placed within or on the external surface of the wall.

1.2—Objective
The objective of this report is to provide guidance in the design and construction of circular prestressed concrete structures circumferentially prestressed using tendons.

1.3—Scope
The recommendations in this report are intended to supplement the general requirements for reinforced concrete and prestressed concrete design, materials and construction, given in ACI 318, ACI 301 and ACI 350R.

This report is concerned principally with recommendations for circular prestressed concrete structures for liquid storage. The recommendations contained here may also be applied to circular structures containing low-pressure gases, dry materials, chemicals, or other materials capable of creating outward pressures. The recommendations may also be applied to domed concrete roofs over other types of circular structures. Liquid storage materials include water, wastewater, process liquids, cement slurry, petroleum, and other liquid products. Gas storage materials include gaseous by-products of waste treatment processes and other gaseous material. Dry storage materials include grain, cement, sugar, and other dry granular products.

The recommendations in this report may also be applicable to the repair of tanks using externally applied tendons.

Design and construction recommendations cover the following elements or components of tendon tanks:
a. Floors
   • Prestressed Concrete
   • Reinforced Concrete
b. Floor-Wall Joints
   • Hinged
   • Fixed
   • Partially Fixed
   • Unrestrained
   • Changing Restraint
c. Walls
   • Cast-in-Place Concrete
   • Precast Concrete
d. Wall-Roof Joints
   • Hinged
   • Fixed
   • Partially Fixed
   • Free
e. Roofs
   • Concrete Dome Roofs with Prestressed Dome Ring
     (1) Cast-in-place Concrete.
     (2) Shotcrete.
   • Other Roofs
     (1) Prestressed Concrete.
     (2) Reinforced Concrete.
f. Wall and Dome Ring Prestressing Methods
   • Circumferential
     (1) Individual high-strength strands in plastic sheaths or multiple high-strength strand tendons in ducts positioned within the wall and post-tensioned after placement and curving of the wall concrete, as shown in Fig. 1.1.
     (2) Individual or multiple high-strength strands and, less frequently, individual high-strength bar tendons, prestressed after being positioned on the exterior surface of the wall.
     • Vertical
       (1) Individual or multiple high-strength strand or individual high-strength bar tendons, enclosed in sheathing or ducts within the wall, anchored near the wall joints at the bottom and top of the wall.
       (2) Pretensioned high-strength strands in precast panels.
1.4—History and development

The late Eugene Freyssinet, a distinguished French engineer generally regarded as the father of prestressed concrete, was the first to recognize the need to use steels of high quality and strength, stressed to relatively high levels, in order to overcome the adverse effects of concrete creep and shrinkage. Freyssinet successfully applied prestressing tendons to concrete structures as early as the late 1920s.

The earliest use of circumferential tendon prestressing in the United States is attributed to the late W. S. Hewett in 1923. He designed and had built several reservoirs using circumferential rods and turnbuckles. A 1932 concrete standpipe in Minneapolis, MN, prestressed by tendons, designed with the Hewett System is still in use and in good condition.

In the early 1950s, following methods used successfully in Europe for a number of years, several circular prestressed concrete tanks were constructed in the United States using post-tensioned high tensile-strength wire tendons embedded in the tank walls. The post-tensioned tendons in most early "tendon tanks" were grouted with a portland cement-water mixture after stressing to help protect them against corrosion and to bond the tendons to the concrete tank walls. Others were unbonded paper-wrapped individual wire or strand tendons that depended on a grease coating and the cast-in-place concrete for their corrosion protection. Later, the use of unbonded tendons with corrosion-inhibiting grease coatings and plastic sheaths became more common. Most of the early tendon tanks constructed in the U.S. followed the common European practice of vertically prestressing the tank walls to eliminate or control horizontal cracking. This crack control helped prevent leakage of the contents and corrosion of the prestressing steel.

Several hundred tendon-stressed tanks (with bonded and unbonded tendons) have been constructed in the United States.

1.5—Definitions

1.5.1 Core wall—That portion of a concrete wall that is circumferentially prestressed. Does not include the shotcrete covercoat in an externally post-tensioned tank.

1.5.2 Joint restraint conditions—Bottom and top boundary conditions for the cylindrical shell wall. Examples are shown in Fig. 1.2 and 1.3.

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

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

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

1.5.2.4 Unrestrained—Limited restraint of radial translation and negligible restraint of rotation (free).

1.5.2.5 Changing restraint—A joint may be of a different type during and after prestressing. An example is a joint that is unrestrained (free) during prestressing but is hinged after prestressing. The change in joint type is a result of grout installation that prevents radial translation after prestressing.

1.5.3 Membrane floor—A thin, highly reinforced, slab-on-grade designed to deflect when the subgrade settles and still retain liquid-tightness.

1.5.4 Shotcrete cover—Pneumatically-applied mortar covering external tendons.

1.5.4.1 Tendon coat—The part of a shotcrete cover in contact with the circumferential prestressing.

1.5.4.2 Body coat—The remainder of the shotcrete cover.
1.5.4.3 **Covercoat**—The tendon coat plus the body coat.

1.5.5 **Tendon**—A steel element such as bar or strand, or a bundle of such elements, used to impart compressive stress to concrete through prestressing. In pretensioned concrete the tendon is the steel element alone. In post-tensioned concrete, the tendon includes the complete assembly consisting of end anchorages and/or couplers, prestressing steel and sheathing or ducts completely filled with a corrosion inhibiting material.

1.5.5.1 **Anchorage**—In post-tensioning, a device used to anchor the tendon to the concrete member.

1.5.5.2 **Bonded tendon**—A prestressing tendon that is bonded to the concrete either directly or through grouting. In a bonded tendon the prestressing steel is not free to move relative to the concrete after stressing and grouting.

1.5.5.3 **Circumferential tendon**—A tendon that is placed around the tank circumference, as shown in Fig. 1.1.

1.5.5.4 **Coupler**—A device used to connect two pieces of a tendon.

1.5.5.5 **Prestressing steel**—High-strength steel used to prestress concrete, commonly seven-wire strands, bars, or groups of strands.

1.5.5.6 **Sheathing**—Enclosures, in which post-tensioning tendons are encased, to prevent bonding during concrete placement and to help protect the strand from corrosion. The enclosures are generally referred to as ducts when used for grouted multiple strand tendons.

1.5.5.7 **Unbonded tendon**—A tendon that is not bonded to the concrete section. In an unbonded tendon the prestressing steel is permanently free to move (between fixed anchorages) relative to the concrete.

1.5.5.8 **Roller**—A short cylindrical segment, usually including a central concave shaped portion, Fig. 1.4, placed under an external tendon to space the prestressed element away from the core wall and reduce friction by rolling along the surface as the tendon is elongated.

### 1.6—Notation

- $A_c$ = area of concrete at cross section considered, sq. in.
- $A_g$ = gross area of unit height of core wall that resists circumferential force due to prestressing, sq. in.
- $A_{gp}$ = gross area of wall that resists externally applied circumferential forces, such as backfill, sq. in.
- $A_{ps}$ = area of prestressed reinforcement, sq. in.
- $A_s$ = area of nonprestressed reinforcement, sq. in.
- $A_{ps} + A_s$ = total area of reinforcement, prestressed plus nonprestressed, sq. in.
- $D$ = dead loads, or related internal moments and forces
- $E_c$ = modulus of elasticity of concrete under short-term load, psi.
- $E_{ci}$ = modulus of elasticity of concrete at age $t_i$, psi.
- $E_s$ = modulus of elasticity of reinforcement, assumed to be the same for prestressed and non-prestressed reinforcement, psi.
- $f'_{c}$ = specified compressive strength of concrete, psi.
- $f'_{ci}$ = specified compressive strength of concrete at time $t_i$ immediately after prestressing (negative for compression), psi.
- $f_{ci}$ = the initial stress in the concrete at time $t_i$ immediately after prestressing, psi.
- $f'_{g}$ = specified compressive strength of shotcrete, psi.
- $f_{pu}$ = specified tensile strength of prestressing strands, wires or bars, psi.
- $f_{re}$ = intrinsic relaxation of prestressed reinforcement that occurs in a tendon stretched between two fixed points (constant strain level equal to initial strain), psi. The intrinsic relaxation depends upon the type and quality of the prestressed reinforcement and the initial prestress level in the steel. Use the prestressing tendon manufacturer’s relaxation data projected to age 50 years. Reference 13 also contains information on this subject.
- $f_y$ = specified yield strength of nonprestressed reinforcement, psi.
- $F$ = loads or related internal moments and forces due to weight and pressures of fluids with well defined densities and controllable maximum heights
- $h$ = thickness of wall, in.
- $h_d$ = thickness of dome shell, in.
- $H$ = loads or related internal moments and forces due to weight and pressure of soil, including water in soil, or stored granular materials
$L = \text{live loads or related internal moments and forces}$

$n_i = \text{initial ratio of elasticity, } E_i / E_c$

$P_w = \text{circular moment force per unit of wall height, lbs., or related internal moments and forces due to the effective circumferential prestressing}$

$P_h = \text{circular moment force per unit of wall height caused by external pressure of soil, ground water in soil, or other loads.}$

$P_l = \text{loads or related internal moments and forces due to the initial circumferential prestressing.}$

$P_n = \text{nominal axial compressive strength of core wall in the circumferential direction per unit of wall height, psi.}$

$P_u = \text{factored unit (uniformly distributed) design load for the dome shell due to dead load and live load, psf.}$

$r = \text{inside radius of tank, ft.}$

$r_d = \text{inside radius of dome, ft.}$

$r_i = \text{averaged maximum radius of curvature over a dome imperfection area with a diameter of 2.5} \frac{r}{2} \pi, \text{ ft.}$

$t = \text{age of concrete at time long term losses are to be calculated, days}$

$t_i = \text{age of concrete at time of prestressing, days}$

$U = \text{required strength to resist factored loads or related internal moments and forces}$

$\beta_i = \text{buckling reduction factor for geometrical imperfections from a true spherical (beta) surface, such as local increases in radius}$

$\beta_c = \text{buckling reduction factor for creep, nonlinearity and cracking of concrete}$

$\Delta P_e = \text{change in compressive force in the concrete, lbs.}$

$\varepsilon_{ss} = \text{free shrinkage strain of concrete. The value of } \varepsilon_{ss} \text{ depends mainly upon the ages } t_j \text{ and } t_i, \text{ the relative humidity and the wall thickness. Values for ultimate shrinkage (in an 8-in. wall between age 14 days and a very long time) recommended by some designers for use in conjunction with the creep coefficients suggested below are } 110 \times 10^{-6}, 260 \times 10^{-6} \text{ and } 420 \times 10^{-6} \text{ for relative humidities of 90, 70 and 40 percent, respectively. As noted below, others recommend higher values for shrinkage and lower values for creep as may be derived from information in ACI 209R.}$

$\eta = \text{aging coefficient for reduction of creep due to prestress loss. A typical value is } \eta = 0.8$

$\eta_r = \text{relaxation reduction factor. A typical value is } \eta_r = 0.8$

$\phi = \text{strength reduction factor}$

$\phi_c = \text{creep coefficient of concrete, defined as the ratio of creep to instantaneous strain. The value of } \phi \text{ depends mainly upon the ages } t_j \text{ and } t_i, \text{ the ambient relative humidity and the wall thickness.}$

Some designers recommend the following coefficients for ultimate creep, after a very long period, in an 8-in. wall prestressed no earlier than age 14 days: 1.6, 2.6 and 2.8 for relative humidities of 90, 70 and 40 percent, respectively. These are used in combination with the values of shrinkage, $\varepsilon_{ss}$, given above. Others recommend lower values of ultimate creep and higher values for shrinkage, as may be derived from information in ACI 209R.

Notes:

A. Units may be inch-pounds or SI, but should be consistent in each equation.

B. Coefficients in equations that contain $\sqrt{f_c}$ or $\sqrt{f_c}$ are for inch-pound units. The coefficient for SI units (MPa) with $\sqrt{f_c}$ and $\sqrt{f_c}$ is the coefficient for inch-pound units divided by 12.

C. Inch-pound units are used in the text. SI conversions are provided in the table in Appendix A.

CHAPTER 2—MATERIALS

2.1—Concrete

2.1.1 General—Concrete should meet ACI 301 and the recommendations of ACI 350R, except as indicated in this report.

2.1.2 Allowable chlorides—For corrosion protection, the maximum water-soluble chloride ion content should not exceed 0.06 percent by weight of the cementitious materials in concrete or grout for prestressed concrete, as determined by ASTM C 1218.

2.1.3 Freezing and thawing exposure—Concrete subject to freezing and thawing cycles should be air-entrained in accordance with ACI 301, Table 4.2.2.4.

2.1.4 Compressive strength—The minimum 28-day compressive strength of any prestressed concrete in tanks should be 4000 psi. In addition, concrete for prestressed floors should reach 1500 psi at 3 days to accommodate two-stage stressing. Nonprestressed footings and roofs may have a 28-day compressive strength as low as 3000 psi.

2.1.5 Water-cement ratio—The water-cement ratio should be 0.45 or less for walls and floors.

2.1.6 Permeability of concrete—It is essential that low-permeability concrete be used for liquid-retaining structures. This can be obtained by using a relatively high cementitious materials content and a low water-cement ratio with high-range water-reducers to help ensure adequate workability. Admixtures such as fly ash, ground-granulated blast-furnace slag and silica fume also decrease permeability. The use of admixtures should follow the recommendations of the suppliers and ACI 212.3R.

2.2—Shotcrete

2.2.1 General—Unless otherwise indicated here, shotcrete should meet ACI 506.2 and the guidelines given in ACI 506R.

2.2.2 Allowable chlorides—Same as for concrete, Section 2.1.2.

2.2.3 Proportioning—Shotcrete should be proportioned in accordance with the following recommendations:

2.2.3.1 The tendon coat should consist of one part portland cement and not more than three parts fine aggregate by weight.

2.2.3.2 The body coat should consist of one part portland cement and not more than four parts fine aggregate by weight.

2.2.3.3 When the covercoat is placed in one application, the mix should consist of one part portland cement and not more than 3 parts fine aggregate by weight.

2.2.4 Compressive strength—The minimum 28-day compressive strength of shotcrete should be 4000 psi.

2.2.5 Freezing and thawing exposure—Dry-mix shotcrete is not recommended for domes in areas subject to freezing and thawing cycles. Wet-mix shotcrete subjected to freezing and thawing cycles should be air-entrained with an in-place air content of 5 percent or greater.

2.3—Admixtures

Admixtures should meet ACI 301 and ASTM C 494. Calcium chloride and other admixtures containing chlorides, fluorides, sulfides and nitrates in more than trace amounts should not be used in prestressed concrete because of potential corrosion problems.

High-range water-reducing admixtures, conforming to ASTM C 494 Type F or G, may be used to facilitate placement of concrete.
2.4—Grout

2.4.1 General—Grout for tendons normally consists of portland cement, water and admixtures and should meet Chapter 18 of ACI 318.

2.4.2 Admixtures—To enhance corrosion protection of the prestressed reinforcement, particularly at tendon high points, portland cement grout for water tank tendons should contain admixtures that lower the water-cement ratio, improve flowability and minimize bleeding. Expansive characteristics may also be provided if desired. The grout, if providing expansion by the evolution of gas, should have 3 to 8 percent total expansion measured in a 20-in. height. An ad-hoc method for determining whether grout is satisfactory is to place the grout in a 1- to 3-in. diameter plexiglass cylinder 25-in. high ten minutes after mixing, cover to minimize evaporation and let it set. No visible bleeding should occur during the test.

2.5—Reinforcement

2.5.1—Nonprestressed reinforcement

2.5.1.2 Strand for wall-to-footing earthquake cables should be epoxy coated (with grit for bond) or galvanized. Epoxy should be fusion bonded, ASTM A 822. Galvanized strand should meet ASTM A 416, Grade 250 or 270, prior to galvanizing; and ASTM A 586, ASTM A 603 or ASTM A 475 after galvanizing. The zinc coating should meet ASTM A 475, Table 4, Class A or ASTM A 603, Table 2, Class A.

2.5.2—Prestressed reinforcement

2.5.2.1 The most common type of prestressed reinforcement used for tendon tanks is stress-relieved, low-relaxation strand. Bars are also used occasionally. Prestressed reinforcement should comply with the recommendations given in this report and with ACI 301. The prestressed reinforcement should also comply with one of the following ASTM designations:

(a) Strands: ASTM A 416 or A 779
(b) Bars: ASTM A 722

2.5.2.2 Both uncoated and galvanized prestressed reinforcement have been used for tendon tanks. Almost all tanks have been constructed with uncoated reinforcement. When galvanized strand or bars are used for prestressed reinforcement, the strand or bars should have a Class A zinc coating as specified in ASTM A 586. The coated strand or bars should meet the minimum elongation of ASTM A 416 or A 722. Epoxy coated strand should meet ASTM A 882.

2.6—Tendon systems

Tendon systems should meet ACI 301, except as indicated here.

2.6.1 Grouted Tendons - Sheathing or duct-forming material should not react with alkalis in the cementitious materials and should be strong enough to retain its shape and resist damage during construction. It should prevent the entrance of cementitious materials slurry from the concrete. Sheathing material left in place should not cause electrolytic action or deterioration. Ducts may be rigid, semi-rigid, or flexible. Ferrous metal and corrugated plastic ducts have been used for tanks. Ducts for grouted tendons should be designed to transfer bond stresses to the adjacent concrete.

2.6.1.1—Ferrous Metal Ducts

(a) Rigid ducts are not normally galvanized by their manufacturer.

(b) Semi-rigid ducts, however, are normally galvanized by their manufacturer, because they are made of a lighter gauge material.

(c) Rigid or semi-rigid ferrous metal ducts typically are used when the prestressing steel is placed in the ducts after the concrete is placed.

2.6.1.2—Corrugated plastic ducts

Corrugated plastic ducts have been used for circumferential and vertical tendons. Corrugated plastic ducts can be continuously watertight if directly connected to the anchorage and properly sealed at couplings. Corrugated plastic ducts should be chemically inert and of adequate thickness and toughness to resist the usual construction wear and tear and radial pressures from curved tendons. Care should be taken to prevent excessive wobble. The ability of the ducts to transfer the desired bond stresses and to resist wear through by radial pressure during stressing should be confirmed by tests.

2.6.2—Unbonded tendons

2.6.2.1 Unbonded tendons typically are used for post-tensioned floors and two-way flat-plate roofs. Unbonded tendons have also been used for vertical wall tendons and, on a less frequent basis, for horizontal circumferential tendons.

2.6.2.2 Prestressing steel, anchorages, sheathing, corrosion preventative coating, and details for providing a complete watertight encapsulation of the prestressing steel, Fig. 2.1, should be in accordance with the Post-Tensioning Institute’s “Specification for Unbonded Single Strand Tendons” for tendons in an aggressive (corrosive) environment. Sheathing should be a high-density polypropylene or polyethylene not less than 60 mils thick, extruded under pressure onto the greased strand, with no space between the inside of the sheathing and the coating material. At the anchorages, the voids in sleeves or caps at the anchorages should be completely filled with corrosion-preventative grease. The sheathing should be connected to all stressing, intermediate and fixed anchorages. This provides complete encapsulation of the prestressing steel from end to end. Connections should remain watertight.

2.6.3—External tendons

2.6.3.1 External tendons are usually spaced away from the wall on rollers or other low-friction supports, Fig. 1.4. They are usually stressed at in-line anchorages or couplers. They may be protected by galvanizing in accordance with Section 2.5.2.2 and 3.1.4.2 (e), by shotcrete in accordance with Sections 3.1.4.2 (e), 4.2.3.5 and 4.5.3.3, or by epoxy in accordance with Section 3.1.4.2 (d).

2.7—Waterstop, bearing pad, and filler materials

2.7.1 Waterstops—Waterstops should be composed of plastic or other suitable materials. Plastic waterstops of polyvinyl chloride meeting CRD-C-572 are recommended. Splices should be made in accordance with the manufacturer’s recommendations. Materials proposed for use on the job
site should be certified by the manufacturer based on laboratory tests, or other tests should be made that will ensure compliance with the specification.

2.7.2 Elastomeric bearing pads—Bearing pads should be composed of neoprene, natural rubber, polyvinyl chloride, or other materials that have demonstrated acceptable performance under similar conditions and applications.

2.7.2.1 Neoprene bearing pads should have a minimum ultimate tensile strength of 1500 psi, a minimum elongation of 500 percent (ASTM D 412), and a maximum compressive set of 50 percent (ASTM D 2240, Type A Durometer). Neoprene bearing pads should comply with ASTM D 2000, Line Call-Out M2BC4105A14B14.

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

2.7.2.3 Polyvinyl chloride for bearing pads should meet the CRD-C-572.

2.7.3 Sponge filler—Sponge filler should be closed-cell neoprene or rubber capable of taking a head of 50 ft. of liquid concrete without absorbing grout and becoming hard. It should also meet ASTM D 1056, Type 2, Class A and Grades 1 through 4. The minimum grade sponge filler recommended for use with cast-in-place concrete walls should be Type 2, Class A and Grade 3.

2.8—Epoxy injection

Epoxy used for injection into cracks, minor honeycomb, separated shotcrete covercoats or wet spots should conform to ASTM C 881, Type I, Grade 1 and should be a two-component, 100-percent-solids, moisture-insensitive epoxy system.

2.9—Epoxy adhesives

Epoxy used for increasing the bond between hardened concrete and plastic concrete should be a two-component, 100-percent-solids, moisture-insensitive epoxy adhesive meeting ASTM C 881, Type II, Grade 2. ACI 503.2 also contains information on this subject. The bonding agent should produce a bond strength (ASTM C 882) not less than 1500 psi 14 days after the plastic concrete is placed.

2.10—Coatings for outer surfaces of tank walls and domes

2.10.1 Above-grade—In some cases, such as tanks located in areas subject to salt spray and landscape sprinklers, coatings may be desired to seal the exterior surface of above-grade shotcrete domes and shotcrete protection for external tendons. Coatings suitable for sealing the exterior of the tank should be permeable to water vapor so as not to trap the higher vapor pressure inside the tank wall. These include polyvinyl chloride-latex and polymeric vinyl-acrylic paints and cementitious materials based coatings.

2.10.2 Below-grade—Coatings are recommended to seal the exterior surface of below-grade tanks that contain dry materials and for protection against aggressive soils. Coatings suitable for sealing the exterior of the tank wall include coal-tar epoxies and bitumastic compounds.

2.10.3 Additional information on coatings for concrete is given in ACI 515.1R.

CHAPTER 3—DESIGN

3.1—Strength and serviceability

3.1.1 General—Structures and components of structures should be designed to provide both the minimum strength and serviceability recommended in this report. Strength and
serviceability recommendations given in this report are intended to ensure adequate safety and performance of structures subject to typical loads and environmental conditions. The control of leakage and protection of embedded steel from corrosion are necessary for adequate serviceability.

3.1.2—Loads and environmental considerations

3.1.2.1—Loads

(a) Prestressing forces—Circumferential prestressing forces in the wall and dome ring, vertical prestressing (if provided in the wall) and roof prestressing that affects the wall, should be considered in the wall design. For example, circumferential prestressing with backfill pressure (when applicable) combines to determine the circumferential compressive strength required. Circumferential prestressing also typically causes vertical bending moments that may add to, and may reduce vertical bending moments from other loading conditions. In these cases load factors other than 1.0 are recommended, as described in Section 3.1.3.

The reduction in prestressing forces with the passage of time due to the inelastic effects of concrete creep, shrinkage and the relaxation of the prestressed reinforcement must be considered.

(b) Internal pressure from stored materials—Fluid pressure in liquid storage vessels, gas pressure in vessels containing gas or materials that generate gas, and lateral pressure from stored granular materials should be considered, as appropriate. Pressure from stored granular material is described in ACI 313.

(c) External lateral earth pressure including the surcharge effects of live and other loads supported by the earth acting on the walls.

(d) Weight of structure.

(e) Wind loads.

(f) Earth, snow, and other live loads on roofs.

(g) External hydrostatic pressure on walls and floors due to ground water.

(h) Seismic effects.

(i) Equipment and piping supported on roofs or walls.

(j) Ice pressure from freezing water in environments where significant amounts of ice form inside tanks. 15, 21

3.1.2.2—Environmental considerations

(a) Thermal and moisture gradients through the thickness of structural elements.

(b) Thermal and moisture gradients along the height of the wall.

(c) Temperature and moisture difference between structural elements.

(d) Exposure to freezing and thawing cycles.

(e) Chemical attack on concrete and metal.

3.1.2.3—Control of loads

(a) 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.

(b) One or more vents should be provided for containment structures. The vents should limit the positive internal pressure to an acceptable level when the tank is being filled at its maximum rate and limit the negative internal pressure to an acceptable level 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 pipe being broken immediately outside of the tank.

(c) Hydraulic pressure-relief valves may be used on non-potable water tanks to control hydrostatic uplift on floor slabs and walls when the tanks are empty or partially full. The use of pressure-relief valves should be restricted to applications where the expected ground-water level is below the operating level of the tank. The valves may also be used to protect the structure during floods. The inlet side of pressure-relief valves should be interconnected with 1) a layer of free-draining gravel adjacent to and underneath the concrete surface to be protected, 2) a perforated pipe drain system placed in free-draining gravel adjacent to the concrete surface to be protected, or 3) a perforated pipe drain system in free-draining gravel that serves as collector system for a geomembrane drain system placed against the concrete surface to be protected.

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

The spacing and size of pressure-relief valves should be adequate to control the hydrostatic pressure on the structure and in general the valves should not be less than 4 in. in diameter or spaced farther than 20-ft. apart. Ideally, the valves or a portion of the valves should be placed at the low point of the structure unless the structure has been designed to withstand the pressure imposed by a ground-water level to, or slightly above, the elevation of the valves.

The use of spring-controlled pressure-relief valves is discouraged because of mechanical problems in the past. Floor-type pressure-relief valves that operate by hydrostatic pressure, and wall-type pressure-relief valves having corrosion-resistant hinges operated by pressure against a flap gate, are recommended. The recommended type of pressure relief valves for floors have covers that are lifted by hydrostatic pressure. They also have restraining lugs that limit the travel of the cover.

Caution should be exercised in using floor-type valves where the operation could be affected by sedimentation within the tank or by incidental contact by a scraper mechanism in the tank. When wall-type valves are used in tanks with scraper mechanisms, the valves should be positioned to clear the operating mechanisms with a flap gate in the opened or closed position, taking into account that there may be some increase in the elevation of the scraper due to buoyancy and/or build-up of sediment on the floor of the tank.

(d) Gas pressure-relief valves should be used to limit gas pressure to acceptable levels on the roofs and walls of non-vented structures such as digester tanks. The type of pressure-relief valve selected should be compatible with the contained gas and the pressure range anticipated. Not less than two valves should be used, at least one valve should be redundant and at least 50 percent redundancy should be pro-
vided. The valve selection should consider any test pressure that may be used on the structure.

(e) Freeboard should be provided in tank walls to minimize earthquake-induced hydrodynamic (sloshing) effects on a flat roof unless a structural analysis shows that freeboard is not needed.

3.1.3 Strength

3.1.3.1 General—Structures and structural members should be proportioned to have strengths that equal or exceed the minimum strength in Chapter 9 of ACI 318, and as recommended in this report.

3.1.3.2 Load factors

(a) The load factors in Chapter 9 of ACI 318 for dead load, live load, wind load, seismic forces, and lateral earth pressure should be used except as noted below. A load factor of 1.7 should be used for lateral pressures from stored solids.

(b) A load factor of 1.5 is recommended for fluid and gas pressure, except the load factor for gas pressure may be reduced to 1.25 for the design of domes with pressure-relief valves.

$$U = 1.5F$$

(c) A load factor of 1.4 should be applied to the final prestress forces (after long term losses) for determination of the circumferential compressive strength of the core wall. For example, when prestress is combined with external soil pressure:

$$U = 1.4P_e + 1.7H$$

(d) Boundary restraints in place at the time of application of the prestressing force, and non-linear distributions of prestressing forces, cause bending moments in walls or other structural components. A load factor of 1.2 should be applied to bending moments produced by the initial prestress force (before long term losses) for cases where the prestress, in combination with other factored loads, produce the maximum flexural strength demands. For example, for bending moments or other effects from initial prestress and external loads that are additive:

$$U = 1.2P_{ei} + 1.7H$$

(e) A load factor of 0.9 should be applied to bending moments produced by the final effective prestress force (after long term losses) for cases where the prestress force reduces the flexural strength needed to resist other factored loads. For example, for bending moments or other effects from internal fluid pressure that are reduced by bending effects from final prestress:

$$U = 0.9P_e + 1.5F$$

3.1.3.3—Design strength

(a) When considering axial load, moment, shear, and torsion, the design strength of a member or cross section should be computed as the product of the nominal strength, calculated in accordance with the provisions of ACI 318, and the applicable strength reduction factor as noted in Chapter 9 of ACI 318, except as follows:

1. Tension in circumferential effective (after losses) prestressing, $\phi = 0.85$
2. Circumferential compression in concrete and shotcrete, $\phi = 0.75$

3.1.4 Serviceability recommendations

3.1.4.1 Watertightness control

(a) Liquid containment structures should be designed to preclude visible flow or leakage (as discussed in Chapter 5) on wall surfaces, as well as leakage at floor-wall connections and through floors and floor joints.

(b) Watertightness acceptance criteria for tanks are given in Chapter 5.

3.1.4.2 Corrosion protection of prestressed reinforcement

(a) Prestressed reinforcement embedded in the concrete is protected by the combination of concrete cover and ducts or sheathing filled with corrosion-inhibiting materials. The minimum concrete covers for tendons, ducts and embedded fittings should not be less than those required by Chapter 7 of ACI 318 and Section 3.1.4.3 of this report.

(b) Bonded post-tensioned tendon reinforcement is normally protected by Portland cement grout.

(c) Unbonded single-strand tendons should be protected by continuous extruded plastic sheathing having a minimum thickness of 0.040 in. The annular space between the sheathing and the strand, as well as the cavities in the anchorages and protective sleeves, should be completely filled with corrosion-inhibiting grease. The tendon protection system should be designed to provide complete encapsulation of the prestressing steel, in addition to the normal concrete cover over the tendon. Patented “electronically isolated” systems that will protect the anchorages from corrosion are also available. References 28 and 29 have information on unbonded tendons in “corrosive environments.”

(d) A minimum of 2 in. of concrete cover is recommended over tendon anchorages and couplers.

(e) Strands having a thermally bonded cross-linked polymer coating for corrosion protection (epoxy-coated strands) are available for use in bonded, and unbonded tendon applications.

(f) External tendons are normally protected with a shotcrete cover. The external tendons should be protected by not less than 1 in. of shotcrete if galvanized or epoxy-coated and 1 1/2 in. if uncoated. Anchorages and couplers should be completely encapsulated in grout and ed by shotcrete. Anchorages and couplers should be protected by not less than 2 in. of shotcrete. Additional shotcrete cover, reinforced with welded wire fabric, may be advisable for external bar tendons.

(g) External tendons not protected by a shotcrete covercoat are not normally recommended. They have occasionally been used, however, for repair of concrete tanks. When used, exposed external tendons should be protected by galvanizing or epoxy coatings along with zinc-rich paint on the exposed anchorage after tensioning. Exposed external tendons should be inspected at frequent intervals and maintained.
ternal tendons are not protected by shotcrete cover, appropriate safety measures should be taken to prevent vandalism.

3.1.4.3 Corrosion protection of nonprestressed reinforcement—Nonprestressed reinforcement should be protected by the concrete cover required in Chapter 7 of ACI 318, except as modified in this Section and in Sections 3.2.1.1 and 3.2.1.2 of this report.

(a) At least 1 in. of concrete cover for corrosion protection is sufficient in two-way post-tensioned walls, roofs and floors exposed to earth, weather, water, or non-aggressive dry materials. At least 1½ in. is recommended for exposure to wastewater. Exposure to aggressive environments may need special consideration.

(b) 1½ in. of concrete cover is recommended for one-way (circumferentially only) post-tensioned walls exposed to earth, weather, water, and wastewater. A minimum of 1 in. of concrete cover is recommended for non-aggressive dry materials. Aggressive materials need special consideration.

3.1.4.4 Boundary conditions—The effects of radial translation and rotation, or the restraint thereof, at the tops and bottoms of tank walls should be included in the analysis of tank walls. The effects of prestressing, external loads, and dimensional changes produced by concrete creep, shrinkage, temperature and moisture content changes should be included in the evaluation of these translations and rotations.

3.1.4.5 Other serviceability recommendations in liquid containment structures—Allowable stresses, provisions for determining prestress losses, bi-directional prestress or reinforcement recommendations that help to preclude leakage, and various other design recommendations intended to ensure serviceability of water tanks and other liquid containment structures, are given in Sections 3.2, 3.3, and 3.4.

3.2—Floor and footing design

3.2.1 Membrane floors—Reinforced concrete membrane floors transmit loads to the subbase without developing significant bending moments. Settlements should be anticipated and provisions made for their effects. Local hard and soft spots beneath the floor, if not avoidable, should be carefully considered in the floor design. Special considerations should be given to floors in tanks founded on more than one type of subbase, such as part cut and part fill.

3.2.1.1 Prestressed concrete membrane floors should not be less than 5 in. thick. An effective prestress of 200 psi after accounting for subgrade friction, including any column or wall footings and construction loads in place at the time of prestressing helps prevent cracking. The prestressing should be combined with conventional reinforcement of 0.0015 times the area of the concrete in each orthogonal direction within the plane of the slab. The prestressed and conventional reinforcement should be alternated within the same planes located within the middle one-quarter of the slab thickness. The tendons should be tensioned as soon as the concrete compressive strength is adequate to resist the anchorage forces. Stressing of the tendons in more than one stage is recommended. Unbonded tendons are typically used for floor prestressing. The maximum recommended spacing of prestressed reinforcement is 24 in.

3.2.1.2 The designer should specify the nonprestressed membrane slab thickness considering the applicable cover provisions of Chapter 7 of ACI 318 and a recognition of the realistic construction tolerances of ACI 117. For crack control, the ratio of nonprestressed reinforcement area to concrete area should not be less than 0.005 in each orthogonal direction in slabs less than 8 in. thick. Section 3.2.5.5 contains recommendations for thickened areas and Section 3.2.1.4 has information on the recommended distribution of nonprestressed reinforcement in thicker slabs. The spacing of reinforcement should not exceed 12 in. for bars and 4 in. for welded wire reinforcement. The reinforcement should be located in the upper portion of the slab thickness, with a minimum cover of 1 in. from the top of the slab and 2 in. from the bottom of the slab (top of the subgrade). Adjacent sheets or rolls of welded wire reinforcement should be overlapped in accordance with ACI 318, but not less than 6 in.

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

3.2.1.4 Conventionally reinforced slabs having a thickness of 8 in. or more should have a minimum reinforcement ratio of 0.006 in each orthogonal direction distributed into two mats. One mat should be located in the upper 3½ in. of the slab thickness, with a minimum cover of 1½ in. from the top of the slab. This mat should provide a minimum ratio of reinforcement area to total concrete area of 0.004 in each orthogonal direction within the plane of the slab. The second mat should be located in the lower 5 in. of the slab with a minimum cover of 3 in. from the top of the subgrade. This mat should provide a minimum ratio of reinforcement area to total concrete area of 0.002 in each orthogonal direction within the plane of the slab. Slabs with a thickness greater than 24 in. need not have reinforcement greater than that recommended for a 24 in. thick slab unless needed to resist loads.

3.2.1.5 Floors subject to hydrostatic uplift pressures that exceed 0.67 times the weight of the floor system should have under-floor drainage or hydrostatic pressure-relief valves to control uplift pressures, or be designed to resist the uplift pressures. Pressure-relief valves should not be used when potable water, petroleum products, or dry materials will be stored in the tanks because of possible contamination of the contents.

3.2.2 Structural floors—Structural floors may be prestressed or nonprestressed. Prestressed structural floors should be designed according to the provisions of ACI 318 except the minimum average prestressing should be 150 psi. Nonprestressed structural floors should be designed using the lower steel stresses or additional load factors of ACI 350R. Structural floors are used when piles or piers are needed to support tank contents because of inadequate soil bearing capacity, expansive subgrade, hydrostatic uplift, or a potential for sinkholes.
3.2.3 Mass concrete—Concrete floors used to counteract hydrostatic uplift pressures may be mass concrete as defined in ACI 116R and ACI 207.1R. Minimum reinforcing recommendations are given in Section 2.2.1.4 of this report. The effect of restraint, volume change and reinforcement on cracking of mass concrete is the subject of ACI 207.2R.

3.2.4 Floor concrete strength—Minimum concrete compressive strengths are recommended in Section 2.1.4.

3.2.5 Floor joints

3.2.5.1 Membrane floors for liquid containment structures should be designed so that the entire floor can be cast without construction joints. If this is not practical, the floor should be designed to minimize construction joints. The construction procedures given in Section 4.1.2 have been effective in minimizing shrinkage cracks and thus producing liquid-tight floors.

3.2.5.2 Waterstops should be provided in joints of floors not having prestressed reinforcement. Separate alignment footings should be provided below the joints or the slab can be thickened at such joints to make room for the waterstop.

3.2.5.3 Waterstops or sealants are used by most designers at construction joints in prestressed floors.

3.2.5.4 Additional nonprestressed reinforcement, up to a total of one percent of the cross-sectional area of the first four feet of the concrete measured perpendicular to the construction joint, should be provided parallel to an existing construction joint in the subsequently placed side of the construction joint, Fig. 3.1. Note that this recommendation only applies to construction joints where the subsequently placed concrete is restrained from shrinkage by deformed bars or dowels that project from the initially placed concrete. This recommendation does not apply to expansion/contraction joints where the subsequently placed concrete is not restrained from shrinking.

3.2.5.5 If the slab is thickened at construction joints or the circumferential edge, any loss of effective prestress in the slab due to the keying effect between the slab and the subgrade should be considered in the design. If the slab is thickened at construction joints, additional reinforcement sufficient to maintain the reinforcing ratios recommended in Section 3.2.1.2 or 3.2.1.2 should be provided parallel to the waterstop. Also, if the slab is thickened at joints, care should be taken to avoid cracks away from the waterstop, such as at the transition to the slab thickness. Whenever the slab is thickened at the perimeter, additional circumferential prestressing or reinforcement, in accordance with Section 3.2.1.1 and 3.2.1.2, should be provided at the thickened slab edge.

3.2.5.6 Floor reinforcement should be continuous through floor joints in tanks with restrained bases. In other tanks, some designers continue the reinforcement through the joints and others have developed details without continuous reinforcement.

3.2.6 Footings

3.2.6.1 A footing should be provided at the base of the wall to distribute vertical and horizontal loads to the subbase. The footing is normally integral with the floor slab.

3.2.6.2 Circumferential prestressed or conventional reinforcement should be provided in the wall footing.

3.2.6.3 The bottom of the footing on the perimeter of a tank should extend at least 12 in. below the adjacent finished grade. A greater depth may be needed for frost protection or for adequate soil bearing.

3.2.6.4 Column footings for tanks are sometimes cast monolithically with the floor slab. If the column footings project below the bottoms of the floor slab, their keying action with the subgrade should be considered in the design. They are designed in accordance with ACI 318. The pressure on the footing from the stored material should be taken into account when evaluating the footing design with respect to the design soil bearing capacity.

3.2.7 Subgrade

3.2.7.1 The subgrade under membrane and mass concrete floors and footings should have sufficient strength and stiffness to support the weight of the tank, its contents and any other loads that might be placed upon it. The subgrade should have sufficient uniformity to control and limit distortion of membrane floors and to minimize differential movement between the footing and the wall.

3.2.7.2 The subgrade soil under floors should be well graded to prevent piping of soil fines out of the subgrade and to remain stable during construction. If the native soils cannot be made acceptable they should be removed and replaced with a properly designed fill.

3.2.8 Floor penetrations

Floor penetrations, such as inlet/outlet pipes, should be detailed to minimize the restraining effects that can occur due to shrinkage and to shortening due to prestressing in post-tensioned concrete floor slabs.

Restraint at improperly detailed slab penetrations can cause cracks in nonprestressed floor slabs and cracks or a reduction of the prestressing forces in prestressed floor slabs. Details that have been used successfully to minimize these effects include concrete closure strips placed after most of the movement has taken place. Flexible seals around the pipe penetrations have also been used successfully to accommodate these movements. Care should be taken in designing these details so the slab will remain watertight, particularly if the pipeline moves due to internal thrust forces or differential settlement in the subgrade soils.

3.3 Wall design

3.3.1 Design methods

3.3.1.1 The design of the wall should be based on elastic cylindrical shell analysis, considering the effects of prestressing, internal loads, backfill and other external loads. The design should also account for:

(a) The effects of friction and anchorage losses, elastic shortening, creep and shrinkage of the concrete, relaxation of prestressed reinforcement, and temperature and moisture gradients.

(b) The joint movements and forces resulting from restraint of deflections, rotations and deformations that are induced by prestressing forces, design loads and dimensional changes.

(c) Variable heights of fluids. Analyses should be performed for the full range of liquid levels between the tank empty and the tank full, to determine the controlling stresses.
3.3.1.2 Coefficients, formulas, and other aids (based on elastic shell analysis) for determining vertical bending moments, circumferential axial and radial shear forces in walls, are given in References 2, 3, 6, 10, 17, and 37.

3.3.1.3 Concrete creep and shrinkage data are provided in ACI 209R.

3.3.1.4 Relaxation data for prestressed reinforcement are given in References 13 and 14.

3.3.2—Wall Details

3.3.2.1 A cast-in-place concrete wall is usually prestressed circumferentially with high-strength strand tendons placed in ducts in the wall. The wall may be prestressed with bonded or unbonded tendons. Vertical prestressed reinforcement near the center of the wall thickness, or vertical nonprestressed reinforcement near each face, may be used. Nonprestressed reinforcement may be provided vertically in conjunction with vertical prestressing.

3.3.2.2 A precast concrete wall usually consists of precast panels curved to the tank radius with joints between panels filled with high-strength concrete. The panels are post-tensioned circumferentially by high-strength strand tendons. The tendons may be embedded within the precast panels or placed on the external surface of the wall and protected by shotcrete, galvanizing or other suitable means. The wall panels may be prestressed vertically with pretensioned strands or post-tensioned tendons. Nonprestressed reinforcement may be provided vertically with or without vertical prestressing.

3.3.2.3—Crack control and liquid-tightness for fluid containment structures

(a) Circumferential prestressing, together with vertical prestressed reinforcement near the center of the wall, or nonprestressed vertical reinforcement near each face of the wall and designed in accordance with Section 3.3.8.2 of this report, aid in crack control and watertightness.

(b) The necessity of obtaining dense, well-compacted concrete, free of honeycombing and cold joints, cannot be overemphasized.

3.3.2.4—Joints in fluid-containment structures

(a) Circumferential (horizontal) construction joints should not be permitted between the base and the top of cast-in-place walls.

(b) Vertical construction joints in cast-in-place concrete walls should contain waterstops and nonprestressed reinforcement passing through the joints to prevent separation of adjacent wall sections prior to prestressing.

(c) Joints between precast concrete wall panels have been constructed with or without waterstops. When waterstops are
omitted the joint surfaces are usually sandblasted prior to placing the concrete or shotcrete closures. The concrete or shotcrete for the closures should be designed to provide at least the same strength as the precast panels. Where vertical joints are small or cold weather conditions make placing conditions adverse, consideration should be given to a higher design strength for the concrete than used for the panels. Shear keys or dowels can be used to prevent radial displacement between precast concrete wall panels prior to prestressing. Shear keys, however, are not structurally necessary and can make the placement of concrete without honeycombing difficult.

3.3.3—Wall proportions

3.3.3.1 Core wall thickness—The core wall thickness should not be less than the following, to facilitate placement of the concrete without segregation.

(a) 10 in. for cast-in-place concrete walls with internal circumferential tendons, with or without vertical tendons, and with conventional reinforcement at the inside or outside faces of the wall.

(b) 9 in. for cast-in-place concrete walls with internal circumferential tendons, and with vertical tendons and conventional reinforcement at or near the center of the wall only.

(c) 8 in. for precast concrete walls with internal circumferential tendons, and with vertical tendons or mats of nonprestressed vertical reinforcement.

(d) 7 in. for precast concrete walls with internal circumferential prestressing and with pretensioned vertical prestressing.

(e) 5 in. for precast concrete walls with external circumferential prestressing and with pretensioned vertical prestressing.

3.3.3.2 Maximum initial prestress—The circumferential compressive stress in the core wall and buttresses produced by the unfactored initial prestress force should not exceed 0.55$f'_c$ for concrete. This stress should be determined based on the net core wall area, after deducting for openings, duct areas and recesses.

3.3.3.3—Circumferential compressive strength

(a) The compressive strength of any unit height of wall for resisting final circumferential prestress force (after friction and long term losses) should be:

$$0.85f'_c\phi[A_g + (2n - 1)A_s] \geq 1.4P_e \quad (3-5)$$

(b) The compressive strength of any unit height of wall for resisting factored external load effects (such as backfill) should be the compressive strength of the wall (including shotcrete protection for external tendons, where applicable) reduced by the core wall strength needed to resist 1.4 times the final circumferential prestress force.

$$\phi(0.85f'_cA_{gr} + A_{ps}f'_s) \left(1 - \frac{1.4P_e}{0.85f'_c[A_g + (2n - 1)A_s]}\right) \geq 1.7P_h \quad (3-6)$$

(c) The wall should also be proportioned so that the maximum compressive axial strain remains within the elastic range under the effects of prestress plus other external loads, such as backfill. The following compressive stress limit is recommended for use in determining minimum wall thickness under final prestress combined with other external effects, such as backfill:

$$\left(\frac{P_c}{A_g + (2n - 1)A_s}\right) + \left(\frac{P_h}{A_{gr} + (2n - 1)A_s + (n - 1)A_{ps}}\right) \leq 0.45f'_c \quad (3-7)$$

For determination of wall circumferential compressive strength, $A_g$ is the gross area of the unit height of core wall at that location. The area of wall recesses, wall penetrations and tendon ducts, however, should be deducted from the wall area in determining $A_g$. An appropriate deduction from $A_g$ should also be made for waterstops. The area of the circumferential prestressing, grout in ducts and shotcrete cover, if any, can be included in the calculation of $A_{gr}$ for backfill or other external loads, $P_h$. When prestressed tanks are repaired by adding tendons, care should be taken to prevent overstressing the walls.

3.3.3.4 For unusual conditions, such as those described in Section 3.3.11, wall thickness should be determined based on a rational analysis, including consideration of wall stability when external loading causes wall compression.

3.3.4 Minimum concrete strength—Minimum specified concrete strength, $f'_c$, given in Section 2.1.4.

3.3.5—Circumferential prestressing

3.3.5.1 The stress in the prestressed reinforcement should not exceed the values specified in Chapter 18 of ACI 318.

3.3.5.2 The circumferential prestressing force should be of sufficient magnitude to:

(a) Counteract axial circumferential tension in the wall due to stored material and other causes after accounting for the prestress losses given in Sections 3.3.5.3 and 3.3.5.4. Backfill should not be considered to counteract internal pressure.

(b) Provide a residual compressive stress of at least 200 psi in the wall, with the tank filled to the design level, after the prestress losses noted in Section 3.3.5.3.

(c) Provide 400 psi at the top of an open top water tank, reducing linearly to not less than 200 psi at 0.6\sqrt{h} below the top of the liquid level. The higher prestress force at the top of open top water tanks has generally been found to be effective in preventing vertical cracking (believed to be caused by temperature and moisture gradients between the wetter and dryer portions of the wall).

(d) The residual compressive stresses recommended above are based on the nominal cross-section of the wall. The actual compressive stress in the concrete is less when the cross sectional area of the nonprestressed steel is accounted for in computing the prestress loss, as described in Section 3.3.5.3 (d).

(e) The residual stress recommended in paragraph (b) is impossible to produce in edge regions that are restrained (prevented from moving inward) during prestressing. There-
fore, restraining the wall base prior to the application of the circumferential prestressing is not recommended without careful consideration of the effects of that restraint. If the wall base is to be restrained at the time of casting, nonprestressed circumferential reinforcement of at least one percent of the cross sectional area of the concrete should be provided to control vertical cracks due to shrinkage and other effects, in that portion of the wall, above the base, where the residual prestressing recommended in 3.3.5.2 (b) is not obtained, as confirmed by analysis, Fig. 3.1. References 4, 11, and 12 provide additional discussion of this subject.

3.3.5.3—Long-term losses in prestressed reinforcement

(a) Calculations for prestress loss due to the long-term effects of creep, shrinkage and steel relaxation for specific applications are preferably made by considering properties of the materials and systems used, the service environment, the load durations, the amount of nonprestressed reinforcement and the stress levels in the concrete and prestressing steel. The calculated losses vary with the assumed long-term average level of contents in the structure. The losses should be calculated for the tank being always full and again for the tank being always empty. The designer can then use judgment as to where to place the long-term losses between these extremes. References 10, 22, 24, 26, 36, and 40 provide additional guidance for calculating long term prestress losses. Reference 40 provides a simple, step-by-step procedure for calculating long-term losses and the information in Reference 10 can be used to estimate the percentage of the total loss that has taken place at any given time.

(b) The prestress losses caused by the long-term effects of creep, shrinkage and steel relaxation in water-containing structures, should not be taken as less than 25,000 psi when normal-relaxation strand or wire is used and 15,000 psi when low-relaxation strand is used. The effect of elastic shortening should be taken into account separately in the calculations (for the tank empty and tank full condition, as applicable).

(c) Prestress losses are generally greater than the values noted above in tanks exposed to low ambient relative humidity, tanks not intended for water storage, or water tanks that remain empty for long periods of time.

(d) In a wall prestressed at age  in, the change in force in the concrete due to creep, shrinkage and relaxation occurring between  and a later time,  may be calculated by:

\[
\Delta P_c = -\beta (\phi_{cs} f'_{cs} A_{cs} + \varepsilon_{cs} A_s E_s + \eta_{rre} A_{ps}) \quad (3-8)
\]

in which

\[
\beta = \left[1 + \frac{n A_{ps}}{A_c}(1 + \eta \phi_{cs})\right]^{-1} \quad (3-9)
\]

\(\Delta P_c\) determined by Eq. 3-8 represents the change in the resultant of stresses on the concrete. Division of \(\Delta P_c\) by \(A_c\) gives the change in stress in concrete due to creep, shrinkage and relaxation. Because of the presence of the nonprestressed steel, \(\Delta P_c\) is not the same as the change in tension in the prestressed steel. Reference 10 contains the derivations of the above equations.

The sign convention used above is: an elongation or a tensile force or stress is positive;  is always negative;  is negative for shrinkage and positive for expansion.

Values of the parameters  and  may be taken from Ghali, 1979 and ACI 209R. More accurate values of the coefficients \(\eta\) and \(\eta_{rre}\) may be determined by graphs or equations in Reference 10.

The following average humidity values may be used in the calculations: 90 percent for a buried water tank; the average between 100 percent and the annual average ambient relative humidity for an above-ground water tank; and the annual average ambient relative humidity for a dry-storage tank. Reference 40 provides guidance on the annual average ambient relative humidity for North America.

3.3.5.4—Friction, seating and elastic shortening losses

(a) Friction, anchorage seating and elastic shortening losses that occur during post-tensioning should be added to the stress loss allowance for creep, shrinkage and steel relaxation, described in Section 3.3.5.3.

(b) Friction losses, including anchorage seating losses, should be calculated in accordance with Chapter 18 of ACI 318. The average stress between adjacent tendons may be used when tendon anchorages are staggered in accordance with Section 3.3.11.2.

(c) Elastic shortening or rebound should be considered as appropriate for the loading condition being investigated, tank empty or full (40).

3.3.5.5—Spacing of prestressed reinforcement

(a) The minimum clear distance between tendons should not be less than 2 in., two times the maximum size of the aggregate, the diameter of duct, or that necessary to limit the tensile stress in the concrete between adjacent ducts due to tendon curvature to 1.2/  whichever is greater.

(b) The maximum center-to-center spacing of circumferential tendons should not exceed three times the wall thickness unless an analysis is made for the effects of greater spacing. The spacing of vertical tendons should not exceed four wall thicknesses, or 4f/ ft., unless vertical nonprestressed reinforcement is provided in regions of flexural tension. In tanks without base restraints (or with base restraints and additional reinforcement) spacings of five or more wall thicknesses have been successfully used.

(c) For unbonded circumferential tendons or bonded circumferential tendons that are widely spaced or have cover exceeding 2 in. from the outer face, consideration should be given to surface crack control due to stresses created by temperature and moisture gradients plus liquid head. Additional prestressed or conventional reinforcement may be needed to control cracking, particularly for unusual climatic or service conditions.

3.3.6 Wall edge restraints and other vertical bending effects

Wall edge restraints, as shown in Fig. 1.2, result in vertical bending moments. Consideration should be given to the following:

3.3.6.1 Interaction—An interaction exists between wall edge restraints, such as restraint of radial translation and rotation, vertical bending moments and hoop forces. The more restraints, especially at wall bases, the greater the vertical bending moments but the lower the hoop forces. Elements
producing restraint should be designed for the resulting restraint forces.

3.3.6.2 Joint details—Various joint details have been devised to minimize discontinuity stresses at tops and bases of tank walls, as shown in Figs. 1.3 and 3.2. These include: 1) joints that incorporate neoprene or rubber pads and other elastomeric materials combined with flexible waterstops to minimize restraint of joint translation and rotation; 2) wall base joints that slide during the application of circumferential prestressing but are subsequently grouted and hinged; and 3) wall base joints that slide during circumferential prestressing and later are provided with closure strips that provide rotational and translational fixity.

(a) Wall restraints at floor—In tanks designed to have restrained bases, restraint of wall base translation and rotation should be delayed for as long as possible after the application of circumferential prestressing. This increases the amount of free movement due to creep and shrinkage that occurs in the highly stressed wall base region before restraints are established.

(b) Wall restraints at roof—The effects of creep, shrinkage and differential moisture and temperature should be considered at the wall-roof joint. Expansion joints (unrestrained) are often used between walls and flat roofs, as shown in Fig. 1.3.

3.3.7—Vertical bending moments

3.3.7.1 Primary vertical bending moments are caused by the following factors and should be considered in wall design,

(a) Internal and external loads in combination with base and top of wall restraints that exist during application of the various loading conditions;

(b) Non-linear distributions of circumferential prestressing;

(c) Banding of prestressing for wall penetrations as described in Section 3.3.9;

(d) Temperature differences between wall, and floor or roof, if restrained; and

(e) Attached structures and pipe restraints (avoid whenever possible).

3.3.7.2 Other factors that can cause secondary bending effects in tank walls should also be considered.

(a) Temperature and moisture gradients through the wall.

(b) Amount and sequence of application of circumferential prestressing.

3.3.8 Design for vertical bending moments—Walls may be vertically reinforced to resist the bending moments described in Sections 3.3.6 and 3.3.7 with prestressed and non-prestressed reinforcement.

3.3.8.1 Prestressed and non-prestressed reinforcement should be proportioned to resist the full flexural tensile stress resulting from bending due to loading conditions in combination with edge restraints, non-linear distributions of circumferential prestressing and other primary bending effects.

Bending moments caused by temperature and moisture gradients through the wall can be unrealistically high if calculated by elastic analysis that ignores creep and cracking. Creep that occurs during the period of temperature or moisture changes reduces the induced stresses. If cracking occurs, the stresses due to temperature and moisture gradients are further reduced.

There is no consensus among experts in tank analysis and design regarding the effects of thermal and moisture gradients through tank walls. Some designers recommend the reduction of these effects that result from an elastic analysis by making relatively liberal assumptions regarding the effective modulus of elasticity, solar radiation, temperature drops at wall surfaces, etc. These designers sometimes also allow minor tensile stresses in the wall provided that the tensile zone
does not penetrate to the reinforcement. This can result in little or no additional vertical reinforcement required for thermal and moisture gradients.

Other designers suggest that cracks produced by thermal and moisture gradients through the wall will be acceptably narrow (not more than 0.004 in.) only when sufficient non-prestressed reinforcement is provided near the faces of the wall. A minimum reinforcing ratio of 0.005 for the total non-prestressed steel, distributed between the two wall faces, has been shown to be effective for this purpose.

Other designers make more conservative assumptions and then provide additional prestressed and non-prestressed reinforcement to account for the relatively high stresses thus calculated. References 10, 27, 32, and 33 offer additional guidance on thermal and moisture gradients.

3.3.9.2 Conventional reinforcement should be proportioned based on the provisions of ACI 350R, except that the maximum allowable tensile stress in the non-prestressed reinforcement should be limited to 18,000 psi. Non-prestressed reinforcement should be provided near wall faces in locations subject to net tensile stress (after allowing for vertical prestressing, if any) from primary bending effects.

3.3.9.3 When vertical prestressing is used, the average vertical stress due to prestressing should be at least 200 psi after friction and long term losses.

3.3.9.4 The combination of vertical prestressed (if any) and non-prestressed reinforcement should also meet the strength recommendations of this report.

3.3.9.5 Walls of structures containing dry materials should be designed for vertical bending effects using non-prestressed or prestressed reinforcement or both in accordance with ACI 318. Special considerations, such as those described in this report for liquid storage tanks, may be helpful if bulk material should be kept dry to prevent expansion or other problems.

3.3.9.6 Pretensioned vertical strands in wall panels need some transfer length before becoming fully effective. Supplemental conventional reinforcement may be necessary in the region of the development length of the strands.

3.3.9 Wall penetrations—Penetrations of walls may be provided for manholes, piping, or other requirements. Seep rings or collars are recommended for tanks containing liquids.

3.3.9.1 Whenever possible, wall penetrations should be located between the designed tendon locations, both circumferential and vertical. When necessary, circumferential tendons may be diverted at an angle (up or down the wall) around penetrations with a minimum horizontal transition distance of 6 times the vertical offset. The design should account for the effects of inclined forces produced by the change in direction of tendon force at the points where tendons are diverted. Consideration should also be given to the additional friction losses produced by these angle changes.

3.3.9.2 The tendon ducts should be located not closer than 2 in. clear to wall penetrations or seep rings (also known as cut-off collars), to prevent seepage along the pipe surface.

3.3.9.3 The wall thickness should be adequate to support the increased circumferential compressive force adjacent to the penetration. Concrete compressive strength may be augmented by compression reinforcement adequately confined by ties in accordance with ACI 318, or by steel edge members around the penetration. The wall thickness may be increased locally, adjacent to the penetration, provided the thickness is changed gradually.

3.3.9.4 Penetrations greater than 2 ft. in height may need special wall designs to ensure adequate reinforcement. Fig. 3.3 shows a special steel collar that has been used effectively for this purpose.

3.3.9.5 Pipes that penetrate walls may need flexible couplings or other means to accommodate differential movements.

3.3.10—Provisions for earthquake-induced forces

3.3.10.1 Tanks should be designed to resist earthquake-induced forces and deformations without collapse or gross leakage. Some designers believe that it is necessary to have some bonded circumferential reinforcement or prestressing for acceptable performance during an earthquake. Design and details should be based upon site-specific response spectra, as well as damping and ductility factors appropriate for the type of tank construction to be used. Alternatively, designs may be based upon static lateral forces intended to account for the effects of seismic risk, damping, construction type and ductility acceptable to the local building official in instances where it is not feasible to obtain site-specific response spectra.

3.3.10.2 Criteria for determining the seismic response of tanks are given in References 1 and 38. Other rational methods for determining the seismic response, such as the energy method are also in use.

3.3.10.3 Sloshing effects of contents, if any, should be considered in the design of walls and roofs.

3.3.10.4 A one third increase in the allowable stresses in the vertical non-prestressed reinforcement is generally considered acceptable when flexural forces include the design earthquake.

3.3.11—Other wall recommendations

3.3.11.1 Special consideration is recommended for unusual conditions. Elastic methods of cylindrical shell analysis, based on the assumption of homogenous, isotropic material behavior, may be employed to evaluate some of the following unusual conditions:

(a) Earth backfill of unequal depth around the tank.
(b) Concentrated loads applied through brackets.
(c) Internally partitioned liquid or bulk storage structures with wall loads that vary circumferentially.
(d) Heavy vertical loads that may affect wall stability.
(e) Large tank radii that may affect wall stability from earth pressure or if externally prestressed;
(f) Containment of hot or cryogenic liquid;
(g) Wind forces on open-top tanks; and
(h) Externally attached appurtenances such as pipes, conduits, architectural treatments, valve boxes, manholes, and miscellaneous structures.

(i) Significant temperature gradients that may affect the core wall during the period after it is cast and before prestressing is applied.
3.3.11.2—Wall buttresses
(a) In order to minimize the effects of friction loss differentials in circumferential tendons of large tanks, as shown in Fig. 1.1, no more than 50 percent of the tendons are typically anchored at any buttress. Hence, except on small tanks, at least four buttresses are generally used. Sometimes more than four buttresses are used to shorten the length and amount of curvature of the individual tendons, thereby reducing friction losses. Alternate circumferential tendons should be anchored at alternate buttresses to provide the most uniform distribution of circumferential prestressing.
(b) For a vertically-prestressed wall, the average vertical prestress in the buttresses should be approximately equal to the average vertical prestress in the wall.
(c) Wall buttresses should be proportioned to avoid reverse curvature of the circumferential tendons unless a specific analysis is made and reinforcement is provided to resist the radial forces resulting from the curvature. A minimum concrete cover of 2 in. or two times the maximum aggregate size, whichever is greater, should be provided over reinforcement and ducts. End anchorages should have a concrete cover of at least 2 in.
(d) The operating area of the jacking equipment used for circumferential prestressing should be considered in the buttress design.
(e) When the end anchorages project outside the buttresses, a continuous vertical concrete cap should be placed to provide at least 2 in. of concrete protection for the end anchorages. The continuous cap should be bonded and anchored into the vertical surface of the buttresses with No. 3 bar (or larger) U-stirrups placed above and below each anchorage. In addition, a No. 5 or larger bar should be placed vertically inside each corner of the tie so that the cap contains a minimum vertical reinforcement area equal to 0.005 times the cap's cross-sectional area.

3.3.11.3 Anchorage zone stresses—Stresses in the anchorage zone can cause splitting and spalling. References 9, 16, 22, 25, 28, 35, and ACI 318 provide guidance on analyzing and designing reinforcements for these stresses.

3.4—Roof design
3.4.1—General
3.4.1.1 Concrete roofs and their supporting columns and footings should be designed in accordance with ACI 318, except for the special provisions given in Section 3.4.2 for dome roofs. The design of nonprestressed concrete roofs over liquid-containing tanks should also be in accordance with the recommendations in ACI 350R.
3.4.1.2 The minimum concrete compressive strength is given in Section 2.1.4.
3.4.2—Dome roofs
3.4.2.1 Design method—Dome roofs should be designed on the basis of elastic shell analysis. References 2, 3, 10, and provide design aids for domes. A circumferentially-prestressed dome ring should be provided at the base of the dome shell to resist the horizontal component of the dome thrust. Unbalanced loads can be significant and require special design procedures, such as finite element techniques.
3.4.2.2 Thickness—Dome shell thickness is governed either by required buckling resistance, by required minimum thickness for practical construction, or by required corrosion protection of reinforcement.
(a) A method for determining the minimum thickness of a monolithic concrete spherical dome shell, to provide adequate buckling resistance, is given in Reference 39. This method is based on the elastic theory of dome shell stability with consideration of the effects of creep, imperfections, and experience with existing tank domes having large radius to thickness ratios. Based on this, the minimum recommended dome thickness is:

\[
\min h_d = r_d \frac{1.5P_u}{\beta_f \beta_c E_c} \quad (3-10)
\]

(b) The conditions that determine the factors \(\beta_f\) and \(\beta_c\) are discussed in Reference 39. The values for these factors, given in subparagraphs 3 and 4, apply for use in Eq. (3-10) when the dome design live load is 12 psf or more, when water is to be stored inside the tank, when the dome thickness is 3 in. or more, when \(f_y\) is 3,000 psi or more, when normal weight aggregates are used, when dead load is applied (that is, shores are removed) not earlier than 7 days after concrete placement, and when curing is per ACI 301. Recommended values for the terms in Eq. (3-10) for such domes are:

1. \(P_u\) is obtained using the minimum load factors given in ACI 318 for dead and live (snow) load.

2. \(\phi = 0.7\) \quad (3-11)

3. \(\beta_f = \left(\frac{r_d}{r_i}\right)^2\) \quad (3-12)

In the absence of other criteria, \(r_i\) may be taken as 1.4\(r_d\) and in this case:

\[
\beta_f = 0.5 \quad (3-13)
\]

4. \(\beta_c = 0.44 + 0.003L\) \quad (3-14)

for live loads between 12 and 30 psf;

5. \(\beta_c = 0.53\) \quad (3-15)

for live loads of 30 psf or greater.

6. \(E_c = 57,000 \sqrt{f_y}\) \quad (3-16)

for normal-weight concrete.

(c) The thickness of precast concrete panel dome shells should not be less than the thickness obtained using Eq. (3-10) when the joints between the panels are equivalent in strength and stiffness to a monolithic shell.

(d) Precast concrete panel domes with joints between panels having lower strength or stiffness than the joint characteristics given in Section 3.4.2.2 (b) may be used if the minimum thickness of the panel is increased above the value given in Eq. (3-10) in accordance with a rational analysis of stability of a dome with a reduced stiffness as a result of the joint details used between adjacent panels.

(e) Other dome configurations, such as cast-in-place or precast domes with ribs cast monolithically with a thin shell, may be used if their design is substantiated by a special analysis. The analysis should show they have adequate strength and buckling resistance for the design live and dead loads with at least the same minimum safety factors established in equation (3-10).

(f) 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 their maximum dead load deflection prior to their final incorporation as a part of the complete dome is greater than 10 percent of their thickness.

(g) The thickness of domes should not be less than 3 in. for monolithic concrete and shotcrete, 4 in. for precast concrete, and 2\(\sqrt{r}\) in. for the outer shell of a ribbed dome.

3.4.2.3 Shotcrete domes—Dry mix shotcrete is not recommended for domes in areas subject to freezing and thawing cycles. Sand lenses caused by overspray and rebound may occur when shooting dry mix shotcrete on relatively flat areas and these are very likely to deteriorate in subsequent freezing and thawing exposures.

3.4.2.4 Reinforcement area—For monolithic domes, the minimum ratio of reinforcement area to concrete area should be 0.0025 in both the circumferential and meridional directions.

(a) In domes with a thickness of 5 in. or less, the reinforcement should be placed approximately at the mid-depth of the shell, except in edge regions. In edge regions of thin domes, and in domes thicker than 5 in., reinforcement should be placed in two layers, one near each face.

(b) Minimum reinforcement may have to be increased for unusual temperature conditions.

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

3.4.2.6 Dome ring—Circular prestressing of the dome ring is employed to eliminate or control the circumferential tension in the dome ring and the dome edge region.

(a) 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 provides for control of shrinkage- and temperature-induced cracking prior to prestressing.

(b) The dome ring reinforcement should have sufficient strength to meet the recommendations of Section 3.1.3.2 for dead and live load factors and Section 3.1.3.3 for strength reduction factors.

(c) An effective prestressing force, after friction and long term losses, should be provided to counteract at least the tension due to dead load, plus a minimum residual circumferential compressive stress equal to the residual compressive stress provided in the wall for dome rings monolithic with the wall or 100 psi for dome rings separated from the wall. Additional prestressing may also be provided to counteract some or all of the live load. If prestressing for less than the
full live load is used, sufficient area of prestressing steel should be maintained at reduced stress, or additional nonprestressed reinforcement should be added, to obtain the strength recommended in Section 3.1.3.

(d) The maximum initial prestress in tendons, after anchoring, should comply with the provisions of Section 3.3.5.1.

(e) The maximum initial compressive stress in dome rings should comply with the provisions of Section 3.3.3.2. Generally, an initial compressive stress of less than 1,000 psi is used in dome rings to limit edge bending moments in regions of the dome and wall (for dome rings not separated from the wall) adjacent to the dome ring.

CHAPTER 4—CONSTRUCTION PROCEDURES

4.1—Concrete

4.1.1 Scope—Procedures for concrete construction should be as specified in ACI 301, except as modified in this report.

4.1.2—Floors

4.1.2.1 Reinforcement should be maintained in its correct vertical position by frequent (4 ft. or less on center each way) support chairs with 2 in. square galvanized or plastic bases or concrete cubes (or equivalent).

4.1.2.2 Concrete in floors should be placed without cold joints and in accordance with the design recommendations in Section 3.2.5. The size and shape of the area to be cast continuously should be selected to minimize construction joints. Factors such as crew size, reliability of concrete supply, time of day and temperature, ACI 302.1R, should be considered to reduce the potential for cold joints during the placing operation.

4.1.2.3 Floors should be cured in accordance with ACI 308. The water curing method (using ponding) is the most commonly used procedure for water tank floors.

4.1.3—Cast-in-place walls

4.1.3.1 A one-to-two-in. layer of neat cement grout is recommended at the base of cast-in-place walls to help preclude voids in this critical area. The grout should have about the same water-cement ratio as the concrete that is used in the wall, and a consistency of thick paint.

4.1.3.2 Some designers recommend a 2-foot thick layer of concrete with \( \frac{1}{4} \text{ in.} \) maximum size aggregate to be placed at the base of the wall to help preclude voids in congested areas such as around vertical prestressing anchorages and waterstops.

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

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

4.1.3.5 Concrete that is honeycombed or does not meet Chapter 18 of ACI 301 should be removed to sound concrete and repaired in accordance with Chapter 9 of ACI 301. An epoxy bonding agent, as described in Section 3.9, should be used when repairing defective areas of water storage tanks.

4.1.4—Precast wall panels and joints

4.1.4.1 Concrete for each panel should be placed in one continuous operation without cold joints or construction joints.

4.1.4.2 Panels should be erected to the correct vertical and circumferential alignment within the tolerances given in Section 4.6.

4.1.4.3 The vertical slots between panels should be free of dirt and foreign substances. Concrete surfaces in the slots should be cleaned and dampened prior to filling. The slots should be filled with cast-in-place concrete, cement-sand mortar or epoxy mortar compatible with the details of the joint. The slot fill should be proportioned, placed, and cured in a manner that will provide the same strength as that specified for concrete in the wall panels as described in Section 2.3.2.4 (e).

4.1.5 Evaluation of concrete—Evaluation of in-situ concrete strength for prestressing, cold-weather conditions, and form removal should be demonstrated by field-cured test cylinders in accordance with ASTM C 873 or pullout strength in accordance with ASTM C 900.

4.2—Shotcrete

4.2.1 Construction procedures—Procedures for shotcrete construction should be as specified in ACI 506.2 and as recommended in 506R, except as modified in this report.

4.2.2 Surface preparation—Prior to application of external prestressed tendons, defects in the core wall should be filled flush with mortar or shotcrete that is bonded to the core wall. Dust, efflorescence, oil, and other foreign materials should be removed after patching defects in the walls. Concrete core walls should be cleaned by abrasive blasting or other suitable means prior to application of prestressed reinforcement and shotcrete. Core walls should have a bondable exterior surface.

4.2.3 Shotcrete cover

4.2.3.1 Externally applied circumferential tendons can be protected by shotcrete cover against corrosion and other damage.

4.2.3.2 The shotcrete cover generally consists of two coats: a tendon coat placed on the prestressed tendons and a body coat placed on the tendon coat. If the shotcrete cover is placed in one coat, the mix should be the same as would be specified for the tendon coat.

4.2.3.3 Tendon coat—The circumferential tendons should be covered first with a tendon coat of cement mortar applied by the pneumatic process as soon as practical after prestressing. Nozzle distance and wetness of mix are equally critical to satisfactory encasement. The shotcrete should be wet, but not dripping, and should provide a minimum cover of \( \frac{1}{4} \text{ in.} \) over the tendon.

(a) The nozzle should be held at a small upward angle, not exceeding 5 degrees, and should be constantly moving, without shaking. It should always be pointing toward the center of the tank. The nozzle distance from the prestressed reinforcement should be such that shotcrete does not build up over or cover the front faces of the tendons until the spaces between them are filled. If the nozzle is held too far back, the shotcrete will deposit on the face of the tendon at the same
time that it is building up on the core wall, thereby not filling the space behind them. This condition is readily apparent and should be corrected immediately by adjusting the nozzle distance and, if necessary, the water content. Care, such as hand dry-packing, should be taken to prevent voids behind larger tendons, such as bar tendons.

(b) After the tendon coat is in place, the wall should be inpected visually to determine whether or not proper encasement has been achieved.

(c) Material placed incorrectly should be removed and replaced.

(d) The tendon coat should be damp cured by a constant spray or trickling of water down the wall, until additional shotcrete is applied to the surface. Curing compounds should not be used on surfaces that will receive additional shotcrete because they interfere with the bonding of subsequent shotcrete layers.

4.2.3.4 Body coat—A body coat should be applied over the tendon coat to complete the minimum specified cover over the outside layer of prestressed reinforcement recommended in Section 2.1.4.2 (e).

(a) If the body coat is not applied as a part of the tendon coat, efflorescence and loose particles should be removed from the surface of the tendon coat prior to the application of the body coat.

(b) Methods of thickness control are suggested in Section 4.2.4.

(c) The completed shotcrete cover should be cured for at least seven days by methods specified in ACI 506.2. Curing should be started as soon as possible without damaging the shotcrete.

4.2.3.5 Special precautions, such as hand dry-packing (not shotcreting), should be taken to prevent voids in and behind anchorages.

4.2.3.6 Separation of the shotcrete cover should not be tolerated. Separation can be detected by “sounding” the exterior surface by tapping it with a hammer after the shotcrete cover has cured. Hollow sounding areas indicate separation. These areas should be eliminated by removal and replacement with properly bonded shotcrete or by epoxy injection.

4.2.4—Thickness control of shotcrete cover for tendons

4.2.4.1 Vertical wires are usually installed to establish uniform and correct thickness of the shotcrete cover. Wires are installed under tension to define the outside surface of the shotcrete from top to bottom. Wires generally are 18- to 20-gage, high-tensile strength steel wire, spaced not more than 36 in. apart circumferentially.

4.2.4.2 The thickness of the shotcrete cover over the tendons should be verified. The following methods may be used. Set screed wires at the surface of the cover or guide wires at a predetermined distance from the tendon surface greater than the cover (for example 2 in., for 1 in. cover) to allow for finishing of the shotcrete surface without interference by the wires. The wires should not be removed until the shotcrete cover thickness has been verified. If the screed or guide wires are no longer in place, the cover thickness may be verified by properly calibrated electronic devices or other methods. If areas are found where the covercoat thickness is less than specified, additional shotcrete should be added to provide the specified thickness.

4.2.5 Cold weather shotcreting—If no housing or other special provision is made for low temperatures, shotcreting may start when the temperature is at least 40 degrees F and rising. Shotcreting should be terminated when the temperature drops to 40 degrees F and is falling. Shotcrete temperatures should be maintained above freezing until it reaches a compressive strength of 500 psi. Shotcrete should not be placed on frozen surfaces. Shotcrete with strength lower than specified due to cold weather should be removed and replaced with sound material.

4.2.6 Evaluation and acceptance of shotcrete strength—Provisions should be made to measure the shotcrete strength, as described in ACI 506.2.

4.3—Forming

4.3.1 Formwork—Formwork should comply with the recommendations of ACI 347R.

4.3.2 Slipforming—Slipforming is not generally used for walls of structures used to contain liquids. This is because of the potential for horizontal cold joints, honeycombing and subsequent leakage.

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, as described in ACI 347R. Ties with snug fitting rubber washers or O-rings have been found to be generally acceptable for this purpose. Tie ends should be recessed in concrete at least 1 in. The holes should be filled with a thoroughly bonded non-corrosive filler at least as strong as the concrete. Taper ties may be used in lieu of ties with waterstops when tapered vinyl plugs and grout are used after casting to fill the voids created by the ties.

4.4—Non-prestressed steel reinforcement

Nonprestressed steel reinforcement should be stored, handled and placed in accordance with ACI 301.

4.5—Prestressing tendons

4.5.1 General—Storing, handling and placing of prestressing tendons should meet ACI 301. Prestressed reinforcement should be stored on dunnage, off the ground, and protected to prevent moisture from unduly (more than a light flake rust) corroding the steel. Under no circumstances should prestressing reinforcement be allowed to stand in ponded water or mud.

4.5.2 Qualifications—All field handling of tendons, and associated stressing and grouting equipment should be under the direction of a person who has technical knowledge of prestressing principles, and qualifying experience (at least 5 years) with the particular system or systems of post-tensioning being used.

4.5.3—Installation

4.5.3.1 Ducts for internal grouted tendons should be fastened securely to prevent distortion, movement or damage from placement and vibration of the concrete. Ducts should be supported to control wobble (consistent with the design parameters). After installation in the forms, the ends of the ducts should be covered to prevent the entry of mortar, water
or debris. Ducts should be inspected prior to concreting to help prevent mortar leakage or indentations that would restrict movement of the prestressed reinforcement during the placing or stressing operation. Where ducts may be subject to freezing prior to grouting, drainage should be provided at any intentional low points to prevent blockage or damage from freezing water. The minimum clear spacing between ducts is given in Section 2.3.5.5 (a).

4.5.3.2 Unbonded monostrand tendons should be tied to supports as necessary to control wobble, but at least every four feet. Care should be taken to prevent tears in sheathing. Breaches in the sheathing should be repaired by proper waterproofing methods.

4.5.3.3 The bars or strands of external multiple-strand tendons that are to be protected by shotcrete cover should be placed in a single layer (not bundled), either directly on the core wall or on rollers or other supports. The minimum clear spacing between strands or bars should be 1.5 diameters of the strands or bars.

4.5.4 Tensioning of tendons—Prestressing tendons are tensioned by means of hydraulic jacks. The effective force in the prestressed reinforcement should not be less than required by the design.

4.5.4.1 Prior to post-tensioning, the prestressed reinforcement should be free and unbonded.

4.5.4.2 Concrete strength at the time of stressing should be at least 1.8 times the maximum initial stress due to the prestressing in any wall section. It should also be sufficient to sustain the concentration of bearing stress under the anchorage plates without damage, per ACI 318. The stressing strength should be confirmed by pullout tests (ASTM C 900) or field-cured cylinders.

4.5.4.3 The vertical tendons, if any, should be tensioned first. Some designers recommend staged stressing, such as stressing every fourth tendon initially, then stressing the remainder. The circumferential tendons should be tensioned in a sequence that will be as symmetrical as practical about the tank’s axis. This generally involves alternating sides of the buttress as tensioning proceeds and alternating buttresses to achieve symmetry. The design prestressing sequence should be detailed on the post-tensioning shop drawings.

4.5.4.4 Tendon elongations calculated by the post-tensioning supplier should be indicated on the shop drawings.

4.5.4.5 The measured elongation of the tendon and the calculated elongations should be resolved in accordance with the provisions of Chapter 18 of ACI 318. Adding the measuring tolerance (about /₁₈ inch), to the normal 7 percent tolerance is considered generally acceptable for short tendons, such as vertical wall tendons.

4.5.5—Grouting

4.5.5.1 Grouted tendons should be grouted as promptly as possible after tensioning. The total exposure time of the prestressing steel to other than a controlled environment prior to grouting should not exceed 30 days, nor seven days after tensioning unless special precautions are taken to protect the prestressing steel. The methods or products used should not jeopardize the effectiveness of the grout as a corrosion inhibitor, nor the bond between the prestressed reinforcement and the grout. Additional restrictions may be appropriate for potentially corrosive environments.

4.5.5.2 Grouting equipment should be capable of grouting at a pressure of 200 psi. However, the tendon ducts should not be over-pressurized during injection if blockage exists. Instead, the grout should be washed out and the blockage removed.

4.5.5.3 Horizontal grouted tendons should have air-release valves, which will also act as standpipes, at intentional high points and drains at intentional low points, such as where tendons are deflected around wall penetrations. These vents and drains, and a vent at the opposite end of the tendon from the point of grout injection, should be closed when a steady stream of pure grout is ejecting. After the vents and drains are closed, the pressure in the duct can be increased to 100 psi to help force the grout into any voids. The pressure should be reduced, but maintained sufficient to prevent backflow, and a valve at the injection end closed to lock off the grout under pressure. If an expansion agent is used, the valves in the stand pipes should then be opened to allow the grout to expand freely. After grout has set, cut off the stand pipes and seal them.

4.5.5.4 Grout injections for vertical tendons should always be from the lowest point in the tendon, to avoid entrapping air. Positive measures should be used at the top to permit free expansion (if an expansive admixture is used) and to accommodate grout settlement. Standpipes 12 in. high have been used at the tops of the ducts or anchorages to allow for grout settlement. When standpipes are used, grout should be wasted until grout flow is free of entrapped air and has the desired consistency. Pumping is then stopped and the standpipe is capped temporarily. After the grout is set, the standpipe should be removed and sealed. Two-stage grouting and admixtures to control bleeding have also been used.

4.5.5.5 The grout should pass through a screen with 0.125-in. maximum clear openings prior to being introduced into the grout pump.

4.5.5.6 To prevent blockages during pumping operations due to the quick setting that can occur in hot weather, either retarders should be added or the grout should be cooled by acceptable methods (such as cooling the mixing water). When freezing weather conditions prevail during and following the placement of grout, adequate means should be provided to protect the grout in the ducts from freezing until the grout attains a minimum strength of 800 psi.

4.5.6—Protection of post-tensioning anchorages

4.5.6.1 Recessed end anchorages in water or other liquid storage tanks should be dry packed with shrinkage-compensating cement mortar. Blockouts in tanks containing dry materials may be dry-packed with a mortar consisting of one part cement to two parts well-graded sand. The minimum cover recommended in Section 2.1.4.2 should be provided.

4.5.6.2 To help ensure bonding, the concrete surfaces, against which concrete encasement over recessed anchorage assemblies is to be placed, should be cleaned. An epoxy bonding agent, as described in Section 3.9, or neat cement grout should be used prior to placing the dry-packed mortar.

4.5.6.3 If continuous vertical concrete caps are placed over the end anchorages of the horizontal tendons, the forms for
these caps should be mortar-tight and fastened solidly to the tank wall and the buttresses to prevent grout leakage. The maximum size aggregate in the concrete should be \( \frac{3}{8} \) in. The concrete should be vibrated to ensure compaction and complete encapsulation around the end anchorages. Concrete cover should be as recommended in Section 2.1.4.2, but not less than 2 inches.

### 4.6—Tolerances

4.6.1 The maximum permissible deviation from the specified tank radius should be 0.1 percent of the inside face radius, or 60 percent of the core wall thickness, whichever is less.

4.6.2 The maximum permissible deviation of the tank inside face radius along any ten feet of circumference should be 5 percent of the core wall thickness.

4.6.3 Walls should be plumb within \( \frac{1}{8} \) in. per 10 feet of vertical dimension.

4.6.4 The wall thickness should not vary more than minus \( \frac{1}{8} \) in. or plus \( \frac{1}{8} \) in. from the specified thickness.

4.6.5 The centers of adjoining precast concrete panels should not vary inwardly or outwardly from one another by more than \( \frac{1}{8} \) in.

4.6.6 To set a limit on the extent of flat areas (Section 2.4.2.1), the surface of the dome is divided into roughly circular areas, each of whose average dimension (measured on the surface of the dome) is \( 2.5 \sqrt{r_p h_d} \). The average radius of curvature of each such area should not exceed \( 1.4 r_d \). The dome may be checked for flat spots by a level survey on the outside surface or by moving a template cut with the proper curvature over the outside surface of the dome.

### 4.7—Seismic cables

When seismic cables, as shown in Fig. 3.2, are installed in floor-wall or wall-roof connections to restrain differential tangential motion between the wall and footing or roof, the following precautions should be taken.

4.7.1 Separation sleeves—Sleeves of rubber or other similar material should surround the strands at the joint. The thickness of the sleeves should be large enough to permit the anticipated radial wall movements. Concrete or grout should be prevented from entering the sleeves. The remainder of the cable should bond to the wall concrete and to the footing concrete.

4.7.2 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 restraint of radial wall movement.

### 4.8—Waterstops and sealants

4.8.1 Placing—Waterstops should be secured by split forms or other means to ensure positive positioning and tied to the reinforcement to prevent displacement during concrete placing operations.

4.8.2 Encasement—Horizontal waterstops should be accessible during concreting. They should be secured in a manner allowing them to be bent up while concrete is placed and compacted underneath, after which they should be allowed to return to position and the additional concrete placed over the waterstop.

4.8.3 Continuity—All waterstops should be spliced in a manner to ensure complete continuity as a water barrier and as recommended by the manufacturer.

4.8.4 Joints with sealants should be constructed to accommodate the calculated movement in accordance with ACI 504R. Joints should be free of form-release agents, loose concrete, moisture, dust, and other contaminants before placing sealants.

### 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 also be held in position and protected from damage from nonprestressed reinforcement by inserting small, dense concrete blocks on top of the pad under the nonprestressed reinforcement ends. Nailing of pads should not be permitted unless pads are specifically designed for such anchorage.

4.9.2 Free sliding joints—When the wall is designed for a wall-floor joint that is free to translate radially, the joint should be detailed and constructed to ensure freedom from obstructions that might prevent 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 at the joint in accordance with the design. 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 pads, between pad and waterstops, and between pad and joint filler, should be filled with non-toxic sealant compatible with the materials of the pad, filler and waterstop, and the concrete surface. No concrete-to-concrete hard spots that would inhibit free translation of the wall should be permitted.

### 4.11—Cleaning and disinfection

4.11.1 Cleaning

4.11.1.1 After the tank has been completed, the interior of the tank should be carefully cleaned out. Rubbish, trash, loose material, and other items of a temporary nature should be removed from the tank. Then the tank should be thoroughly cleaned with a high-pressure water jet, sweeping, scrubbing, or equally effective means. Water and dirt or foreign material accumulated in this cleaning operation should be discharged from the tank or otherwise removed. The interior surfaces of the tank should be kept clean until final acceptance.

4.11.1.2 Following the cleaning operation, the vent screen, overflow screen, and any other screened openings should be checked and put in satisfactory condition to prevent birds, in-
sects and other possible contaminants from entering the facility.

4.11.2 Disinfection—Potable water tanks should be disinfected in accordance with AWWA C652.

CHAPTER 5—ACCEPTANCE CRITERIA FOR LIQUID-TIGHTNESS OF TANKS

5.1—Testing

5.1.1 General—A test for watertightness should be performed on tanks intended for water storage. Similar liquid-tightness tests should be made for tanks intended for storage of liquids other than water. Tanks intended for storage of dry materials need not be tested for watertightness.

5.1.2 Watertightness testing—The test should be made over a period of at least 24 hours with a full tank. Alternatively, the following time periods for the watertightness test (based on ACI 350.1R) may be used. Maintain the tank full for three days (72 hours) prior to beginning the test. Measure the drop in liquid level over the next three to five days to determine the daily average for comparison with the acceptance criteria given in Section 5.2.

5.2—Acceptance criteria

5.2.1 Watertightness—In tanks intended for storage of potable or raw water, the loss of water in a 24-hour period should not exceed 0.05 percent of the tank volume. If the loss of water exceeds 0.025 percent of the tank volume the tank should be inspected for point sources of leakage. If point sources are found they should be repaired.

5.2.2 Special conditions—In soils subject to piping action or swelling, or where the contents of the tank would have an adverse environmental impact, more stringent criteria than the limit of Section 5.2.1 may be appropriate. References 1, 5, and 8 and ACI 350.2R provide additional guidance for tanks where additional liquid-tightness is desired, and for tanks containing municipal and industrial sewage, petroleum products and hazardous wastes.

5.3—Visual criteria

5.3.1 Seepage—Seepage that produces moisture on the wall that can be picked up on a dry hand or facial tissue should not be accepted. External tendon tanks with shotcrete covercoats are normally checked for watertightness prior to application of the shotcrete covercoat.

5.3.2 Visible flow—Visible flow of tank contents from beneath the tank should not be permitted.

5.3.3 Floor-wall joint—Visible flow of the tank contents through the wall-floor joint should not be permitted. Dampness on top of the footing, that cannot be observed to be flowing, is acceptable.

5.3.4 Ground water—Floors, walls and wall-floor joints should not allow ground water into the tank.

5.4—Repairs and retesting

5.4.1 Repairs—Repairs should be made if the tank fails the watertightness test, including the visual criteria, or is otherwise defective. The materials of repair should be in accordance with Section 2.8 for epoxy injection, Section 3.9 and ACI 301 for patched areas, or other acceptable materials. The methods of repair should be in accordance with the requirements of ACI 301.

5.4.2 Retesting—After repair, the tank should be retested to confirm that it meets the watertightness criteria.

CHAPTER 6—REFERENCES

6.1—Recommended references

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designation. Since some of these documents are revised frequently, the user of this document should check directly with the sponsoring group if it is desired to refer to the latest version.

American Concrete Institute (ACI)

116 R Cement and Concrete Terminology
117 Standard Specifications for Tolerances for Concrete Construction and Materials
212.3 R Chemical Admixtures for Concrete
207.1 R Mass Concrete for Dams and Other Massive Structures
207.2 R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete
209 R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
301 Specifications for Structural Concrete
302.1 R Guide for Concrete Floor and Slab Construction
304 R Guide for Measuring, Mixing, Transporting, and Placing Concrete
308 Standard Practice for Curing Concrete
313 Standard Practice for Design and Construction of Concrete Silos and Stacking Tubes for Storing Granular Materials
318 Building Code Requirements for Reinforced Concrete
347 R Guide for Formwork for Concrete
349 Code Requirements for Nuclear Safety Related Concrete Structures
350 R Environmental Engineering Concrete Structures
350.1 R Testing Reinforced Concrete Structures for Watertightness
350.2 R Concrete Structures for Containment of Hazardous Materials
503.2 Standard Specification for Bonding Plastic Concrete to Hardened Concrete with a Multi-Component Epoxy Adhesive
504 R Guide to Sealing Joints in Concrete Structures
506 R Guide to Shotcrete
506.2 Shotcrete Specifications
515.1 R A Guide to the Use of Waterproofing, Dampproofing, Protective and Decorative Barrier Systems for Concrete

American Society for Testing and Materials (ASTM)

A 416 Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
A 475 Specification for Zinc-Coated Steel Wire Strand
A 822 Specification for Seamless, Cold Drawn Carbon Steel Tubing for Hydraulic System Service
A 586 Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand
A 603 Specification for Zinc-Coated Steel Structural Wire Rope
A 722 Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete
A 779 Specification for Steel Strand, Seven Wire, Uncoated, Compacted, Stress-Relieved for Prestressed Concrete
A 882 Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand
C 494 Specification for Chemical Admixtures for Concrete
C 873 Standard Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds
C 881 Specification for Epoxy-Resin-Base Bonding Systems for Concrete
C 882 Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete
C 900 Test Method for Pullout Strength of Hardened Concrete
C 1218 Standard Test Method for Water-Soluble Chloride in Mortar and Concrete
D 395 Test Methods for Rubber Property-Compression Set
D 412 Test Methods for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers-Tension
D1056 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 (AWWA)
C 652 Disinfection of Water Storage Facilities

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

The above publications may be obtained from the following organizations:

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

American Society for Testing and Materials
100 Barr Harbor Dr.
W. Conshohocken, PA 19428-2959

American Water Works Association
6666 West Quincy Avenue
Denver, Colorado 80235

U.S. Army Corps of Engineers Specifications
Superintendent of Documents
U.S. Government Printing Office
Washington, DC 20402

6.2—Cited references

21. Kong, W. L., and Campbell, T. L., Thermal Pressure Due to an Ice Cap in an Elevated Water Tank, Department of Civil Engineering, Queen’s University, Kingston Ontario, Canada, revised Mar. 20, 1987.
27. Portland Cement Association, “Circular Concrete Tanks Without
Prestressing,” Information Sheet IS072D, Skokie, 32 pp.
30. Precast/Prestressed Concrete Institute, Recommended Practice for Design and Construction of Precast/Prestressed Concrete Tanks, Chicago.
31. Precast/Prestressed Concrete Institute, State of the Art for Precast/Prestressed Concrete Tanks, Chicago.
APPENDIX A

Conversion factors: Inch-pounds to SI (metric)\(^1\)

<table>
<thead>
<tr>
<th>To convert from</th>
<th>to</th>
<th>Multiply by(^2)</th>
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<tbody>
<tr>
<td><strong>Length</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>inch</td>
<td>millimeter (mm)</td>
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</tr>
<tr>
<td>foot</td>
<td>meter (m)</td>
<td>0.3048E</td>
</tr>
<tr>
<td>yard</td>
<td>meter (m)</td>
<td>0.9144E</td>
</tr>
<tr>
<td>mile (statute)</td>
<td>kilometer (km)</td>
<td>1.609</td>
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<tr>
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<tr>
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<td>square millimeter (mm(^2))</td>
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</tr>
<tr>
<td>square foot</td>
<td>square meter (m(^2))</td>
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</tr>
<tr>
<td>square yard</td>
<td>square meter (m(^2))</td>
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<tr>
<td><strong>Volume (capacity)</strong></td>
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<tr>
<td>ounce</td>
<td>cubic millimeter (mm(^3))</td>
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</tr>
<tr>
<td>gallon</td>
<td>cubic meter(^3) (m(^3))</td>
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</tr>
<tr>
<td>cubic yard</td>
<td>cubic meter (m(^3))</td>
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</tr>
<tr>
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</tr>
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<td>4.448</td>
</tr>
<tr>
<td>pound-force</td>
<td>newton (N)</td>
<td>4.448</td>
</tr>
</tbody>
</table>

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1. This selected list gives practical conversion factors of units found in concrete technology. The reference sources for information on SI units and more exact conversion factors are ASTM E 621 and ASTM E 380. Symbols of metric units are given in parenthesis.

2. “E” indicates that the number is exact.

3. One liter (cubic decimeter) equals 0.001 m\(^3\) or 1000 cm\(^3\).