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Guide for the Analysis, Design, and Construction of
Concrete-Pedestal Water Towers

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This ACI guide presents recommendations for materials, analysis, design, and construction of concrete-pedestal elevated water storage tanks. These structures are commonly referred to as composite-style elevated water tanks that consist of a steel water storage tank supported by a cylindrical reinforced concrete-pedestal. This document includes determination of design loads, and recommendations for design and construction of the cast-in-place concrete portions of the structure.

Concrete-pedestal elevated water-storage tanks are structures that present special problems not encountered in typical building designs. This guide refers extensively to ACI 318 Building Code Requirements for Structural Concrete for many requirements, and describes how to apply ACI 318 to these structures. Determination of snow, wind, and seismic loads based on ASCE 7 is included. These loads will conform to the requirements of national building codes that use ASCE 7 as the basis for environmental loads. Special requirements, based on successful experience, for the unique aspects of loads, analysis, design and construction of concrete-pedestal tanks are presented.

Keywords: analysis; composite tanks; concrete-pedestal tanks; construction; design; earthquake resistant structures; elevated water tanks; formwork (construction); loads (forces): dead, live, water, snow, wind and earthquake loads; load combinations; shear; shear strength; structural analysis; structural design; walls.

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Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/Engineer.
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CHAPTER 1—GENERAL
1.1—Introduction
The objective of this document is to provide guidance for those responsible for specifying, designing, and constructing concrete-pedestal elevated water-storage tanks. Elevated tanks are used by municipalities and industry for potable water supply and fire protection. Commonly built sizes of concrete-pedestal water tanks range from 100,000 to 3,000,000 gallons (380 to 11,360 m³). Typical concrete support structure heights range from 25 to 175 ft (7.5 to 53 m), depending on water system requirements and site elevation. The interior of the concrete support structure may be used for material and equipment storage, office space, and other applications.

1.2—Scope
This document covers the design and construction of concrete-pedestal elevated water tanks. Topics include materials, construction requirements, determination of structural loads, design of concrete elements including foundations, geotechnical requirements, appurtenances, and accessories.

Designs, details, and methods of construction are presented for the types of concrete-pedestal tanks shown in Fig. 1.2. This document may be used in whole or in part for other tank configurations, however, the designer should determine the suitability of such use for other configurations and details.

1.3—Drawings, specifications, and calculations
1.3.1 Drawings and Specifications—Construction documents should show all features of the work including the size and position of structural components and reinforcement, structure details, specified concrete compressive strength, and the strength or grade of reinforcement and structural steel. The codes and standards to which the design conforms, the tank capacity, and the design basis or loads used in design should also be shown.

1.3.2 Design Basis Documentation—The design coefficients and resultant loads for snow, wind and seismic forces, and methods of analysis should be documented.

1.4—Terminology
The following terms are used throughout this document. Specialized definitions appear in individual chapters.

Appurtenances and accessories—Piping, mechanical equipment, vents, ladders, platforms, doors, lighting, and related items required for operation of the tank.

Concrete support structure—Concrete support elements above the top of the foundation: wall, ringbeam, and dome or flat slab tank floor.

Construction documents—Detailed drawings and specifications conforming to the project documents used for fabrication and construction.

Foundation—The concrete annular ring, raft, or pier cap.

Project documents—Drawings, specifications, and general terms and conditions prepared by the specifier for procurement of concrete-pedestal tanks.

Intermediate floor slabs—One or more structural floors above grade, typically used for storage.

Rustication—Shallow indentation in the concrete surface, formed by shallow insert strips, to provide architectural effect on exposed surfaces, usually 7/4 in. (20 mm) deep by 3 to 12 in. (75 to 300 mm) wide.

Ringbeam—The concrete element at the top of the wall, connecting the wall and dome, and the support for the steel tank cone.

Wall or support wall—The cylindrical concrete wall supporting the steel tank and its contents, extending from the foundation to the ringbeam.

Tank floor—A structural concrete dome, concrete flat slab, or a suspended steel floor that supports the tank contents inside the support wall.

Steel liner—A non-structural welded steel membrane placed over a concrete tank floor and welded to the steel tank to provide a liquid tight container; considered a part of the steel tank.

Steel tank—The welded steel plate water containing structure comprised of a roof, side shell, conical bottom section outside the support wall, steel liner over the concrete tank floor or a suspended steel floor, and an access tube.

Slab-on-grade—Floor slab inside the wall at grade.

1.5—Notation
1.5.1 Loads—The following symbols are used to represent applied loads, or related forces and moments; Sections 4.3.3 and 4.4.2.
1.5.2 Variables—The following symbols are used to represent variables. Any consistent system of measurement may be used, except as noted.

- \( D \) = dead load
- \( E \) = horizontal earthquake effect
- \( E_v \) = vertical earthquake effect
- \( F \) = stored water
- \( G \) = eccentric load effects due to dead load and water
- \( L \) = interior floor live loads
- \( S \) = larger of snow load or minimum roof live load
- \( T \) = force due to restrained thermal movement, creep, shrinkage, or differential settlement
- \( W \) = wind load effect

- \( A \) = effective concrete tension area, in.\(^2\) (mm\(^2\)); Section 4.4.3
- \( A_{cv} \) = effective horizontal concrete wall area resisting factored in-plane shear \( V_{gs} \) in.\(^2\) (mm\(^2\)); Section 4.8.6
- \( A_f \) = horizontal projected area of a portion of the structure where the wind drag coefficient \( C_f \) and the wind pressure \( p_z \) are constant; Section 4.6.3
- \( A_g \) = gross concrete area of a section
- \( A_s \) = area of nonprestressed tension reinforcement
- \( A_v \) = effective peak velocity-related ground acceleration coefficient; Section 4.7.4
- \( A_w \) = gross horizontal cross-sectional concrete area of wall, in.\(^2\) (mm\(^2\)) per unit length of circumference; Section 4.8.3
- \( b \) = width of compression face in a member
- \( b_d \) = width of a doorway or other opening; Section 4.8.5
- \( b_e \) = combined inside and outside base plate edge distances; Section 4.10.5

Fig 1.2—Common configurations of concrete-pedestal tanks
1.6 —Metric units

The in.-lb system is the basis for units of measurement in this guide, and soft metric conversion is shown in parentheses.

CHAPTER 2—MATERIALS

2.1—General

Materials and material tests should conform to ACI 318, except as modified in this document.

2.2—Cements

Cement should conform to ASTM C 150 or C 595, excluding Types S and SA, which are not intended as principal cementing agents for structural concrete. The same brand and type of cement should be used throughout the construction of each major element.
2.3—Aggregates
Concrete aggregates should conform to ASTM C 33 and ACI 318. Aggregates used in the concrete support wall should be suitable for exterior exposed surfaces. Where sandblasting or other finishing techniques that expose aggregate are used, the fine and coarse aggregate should be from a consistent source to maintain uniformity of color.

2.4—Water
Water should conform to ASTM C 94.

2.5—Admixtures
Admixtures should conform to ACI 318.

2.6—Reinforcement
2.6.1 Bar reinforcement—Deformed bar reinforcement should conform to ASTM A 615/A 615M, A 617/A 617M, or A 706/A 706M.
2.6.2 Welded wire reinforcement—Welded wire reinforcement should conform to ASTM A 185 or A 497.

CHAPTER 3—CONSTRUCTION

3.1—General
3.1.1 Reference Standard—Concrete, formwork, reinforcement, and details of the concrete support structure and foundations should conform to the requirements of ACI 318, except as modified in this document.
3.1.2 Quality Assurance—A quality assurance plan to verify that the construction conforms to the design requirements should be prepared. It should include the following:
   (a) Inspection and testing required, forms for recording inspections and testing, and the personnel performing such work;
   (b) Procedures for exercising control of the construction work, and the personnel exercising such control;
   (c) Methods and frequency of reporting, and the distribution of reports.

3.2—Concrete
3.2.1 General—Concrete mixtures should be suitable for the placement methods, forming systems and the weather conditions during concrete construction, and should satisfy the required structural, durability and architectural parameters.
3.2.2—Concrete quality
3.2.2.1 Water-cementitious material ratio—The water-cementitious material ratio should not exceed 0.50.
3.2.2.2 Specified compressive strength—The minimum specified compressive strength of concrete should conform to the following:
   (a) concrete support structure = 4000 psi (28 MPa);
   (b) foundations and intermediate floors = 3500 psi (24 MPa);
   (c) slabs-on-grade (see Table 5.8.2).
3.2.2.3 Air-entrainment—Concrete should be air-entrained in accordance with ACI 318.
3.2.3 Proportioning—Proportioning of concrete mixtures should conform to the requirements of ACI 318 and the procedure of ACI 211.1.
3.2.3.1 Workability—The proportions of materials for concrete should be established to provide adequate workability and proper consistency to permit concrete to be worked readily into the forms and around reinforcement without excessive segregation or bleeding for the methods of placement and consolidation employed.
3.2.3.2 Slump—The slump of concrete provided should be based on consideration of the conveying, placing and vibration methods as well as the geometry of the component, and should conform to the following:
   (a) Concrete without high-range water-reducing admixtures (HRWRA) should be proportioned to produce a slump of 4 in. (100 mm) at the point of placement.
   (b) Slump should not exceed 8 in. (200 mm) after addition of HRWRA, unless the mix has been proportioned to prevent segregation at higher slump.
   (c) The slump of concrete to be placed on an inclined surface should be controlled such that the concrete does not sag or deform after placement and consolidation.
3.2.3.3 Admixtures—Admixtures may be used to achieve the required properties. Admixtures should be compatible such that their combined effects produce the required results in hardened concrete as well as during placement and curing.
3.2.4 Concrete production—Measuring, mixing and transporting of concrete should conform to the requirements of ACI 318 and the recommendations of ACI 304R.
3.2.4.1 Slump adjustment—Concrete that arrives at the project site with slump below that suitable for placing may have water added within limits of the slump and permissible water-cementitious material ratio of the concrete mix. The water should be incorporated by additional mixing equal to at least half of the total mixing time required. No water should be added to the concrete after plasticizing or high-range water-reducing admixtures have been added.
3.2.5 Placement—Placing and consolidation of concrete should conform to ACI 318, and the recommendations of ACI 304R and ACI 309R.
3.2.5.1 Depositing and consolidation—Placement should be at such a rate that the concrete that is being integrated with fresh concrete is still plastic. Concrete that has partially hardened or has been contaminated by foreign materials should not be deposited. Consolidation of concrete should be with internal vibrators.
3.2.5.2 Support wall—Drop chutes or tremies should be used in walls and columns to avoid segregation of the concrete and to allow it to be placed through the cage of reinforcing steel. These chutes or tremies should be moved at short intervals to prevent stacking of concrete. Vibrators should not be used to move the mass of concrete through the forms.
3.2.6 Curing—Curing methods should conform to ACI 318 and the requirements of ACI 308. Curing methods should be continued or effective until concrete has reached 70 percent of its specified compressive strength $f'_c$ unless a higher strength is required for applied loads. Curing should commence as soon as practicable after placing and finishing. Curing compounds should be membrane forming or combination curing/surface hardening types conforming to ASTM C 309.
3.2.7—Weather
3.2.7.1 Protection—Concrete should not be placed in rain, sleet, snow, or extreme temperatures unless protection...
is provided. Rainwater should not be allowed to increase mixing water nor to damage surface finish.

3.2.7.2 Cold weather—During cold weather, the recommendations of ACI 306 should be followed.

3.2.7.3 Hot weather—During hot weather the recommendations of ACI 305R should be followed.

3.2.8 Testing, evaluation and acceptance—Material testing, type and frequency of field tests, and evaluation and acceptance of testing should conform to ACI 318.

3.2.8.1 Concrete strength tests—At least four cylinders should be molded for each strength test required. Two cylinders should be tested at 28 days for the strength test. One cylinder should be tested at 7 days to supplement the 28-day tests. The fourth cylinder is a spare to replace or supplement other cylinders. Concrete temperature, slump, and air content measurements should be made for each set of cylinders. Unless otherwise specified in the project documents, sampling of concrete should be at the point of delivery.

3.2.8.2 Early-age concrete strength—Where knowledge of early-age concrete strength is required for construction loading, field-cured cylinders should be molded and tested, or one of the following non-destructive test methods should be used when strength correlation data are obtained:

(a) Penetration resistance in accordance with ASTM C 803;
(b) Pullout strength in accordance with ASTM C 900;
(c) Maturity-factor method in accordance with ASTM C 1074.

3.2.8.3 Reporting—A report of tests and inspection results should be provided. Location on the structure represented by the tests, weather conditions, and details of storage and curing should be included.

3.2.9—Joints and embedments

3.2.9.1 Construction joints—The location of construction joints and their details should be shown on construction drawings. Horizontal construction joints in the support wall should be approximately evenly spaced. The surface of concrete construction joints should be cleaned and laitance removed.

3.2.9.2 Expansion joints—Slabs-on-grade and intermediate floor slabs not structurally connected to the support structure should be isolated from the support structure by premolded expansion joint filler.

3.2.9.3 Contraction joints—Contraction joints are only used with slabs-on-grade (see Section 5.8.2.3).

3.2.9.4 Embedments—Sleeves, inserts, and embedded items should be installed prior to concrete placement, and should be accurately positioned and secured against displacement.

3.3—Formwork

3.3.1—General

Formwork design, installation, and removal should conform to the requirements of ACI 318 and the recommendations of ACI 347R. Formwork should ensure that concrete components of the structure will conform to the correct dimensions, shape, alignment, elevation and position within the established tolerances. Formwork systems should be designed to safely support construction and expected environmental loads, and should be provided with ties and bracing as required to prevent the leakage of mortar and excessive deflection.

3.3.1.1 Facing material—Facing material of forms used above finished grade should be metal, or plywood faced with plastic or coated with fiberglass. Any form material may be used for below-grade applications.

3.3.1.2 Chamfers—Exposed corners should be formed with chamfers 3/4 in. (20 mm) or larger.

3.3.1.3 Concrete strength—The minimum concrete compressive strength required for safe removal of any supports for shored construction, or the safe use of construction embedments or attachments should be shown on construction drawings, or instructions used by field personnel.

3.3.1.4 Cleaning and coating—Form surfaces should be cleaned of foreign materials and coated with a non-staining release agent prior to placing reinforcement.

3.3.1.5 Inspection—Prior to placing concrete, forms should be inspected for surface condition, accuracy of alignment, grade and compliance with tolerance, reinforcing steel clearances and location of embedments. Shoring and bracing should be checked for conformance to design.

3.3.2—Foundations

3.3.2.1 Side forms—Straight form panels that circumscribe the design radius may be used to form circular foundation shapes. Circular surfaces below final ground level may have straight segments that do not exceed 30 deg of arc, and surfaces exposed to view may have straight segments that do not exceed 15 deg of arc.

3.3.2.2 Top forms—Forms should be provided on top sloping surfaces steeper than 1 vertical to 2.5 horizontal, unless it can be demonstrated that the shape can be adequately maintained during concrete placement and consolidation.

3.3.2.3 Removal—Top forms on sloping surfaces may be removed when the concrete has attained sufficient strength to prevent plastic movement or deflection. Side forms may be removed when the concrete has attained sufficient strength such that it will not be damaged by removal operations or subsequent load.

3.3.3—Support wall

3.3.3.1 Wall form—The support wall should be constructed using a form system having curved, prefabricated form segments of the largest practical size in order to minimize form panel joints. Formwork should be designed for lateral pressures associated with full height plastic concrete head. Bracing should be provided for stability, construction related impact loading, and wind loads. Working platforms that allow access for inspection and concrete placement should be provided.

3.3.3.2 Deflection—Deflection of facing material between studs as well as studs and walers should not exceed 1/400 times the span during concrete placement.

3.3.3.3 Rustications—A uniform pattern of vertical and horizontal rustications to provide architectural relief is recommended for exterior wall surfaces exposed to view. Construction joints should be located in rustications.

3.3.3.4 Form ties—Metal form ties that remain within the wall should be set back 11/2 in. (40 mm) from the concrete surface.
3.3.3.5 Removal—Vertical formwork not supporting the weight of the component may be removed when the concrete has reached sufficient strength such that it will not be damaged by the removal operation and subsequent loads.

3.3.4 Tank floor

3.3.4.1 Design—Formwork for the flat slab or dome tank floor should be designed to support construction loads including weight of forms, plastic concrete, personnel, equipment, temporary storage, and impact forces. Unsymmetrical placement of concrete should be considered in the design. Camber to offset concrete weight should be provided where deflection would result in out-of-tolerance construction.

3.3.4.2 Removal—Forms should remain in place until the concrete has gained sufficient strength not to be damaged by removal operations and subsequent loads. The minimum required concrete strength for form removal should be shown on construction drawings or instructions issued to the field.

3.4 Reinforcement

3.4.1 General—Reinforcement should be clearly indicated on construction drawings and identified by mark numbers that are used on the fabrication schedule. Location, spacing as well as lap splice lengths of reinforcement, and concrete cover should be shown. Symbols and notations should be provided to indicate or clarify placement requirements.

3.4.2 Fabrication—The details of fabrication, including hooks and minimum diameter of bends, should conform to the requirements of ACI 318 and ACI 315.

3.4.3 Placement—Reinforcement should be accurately positioned, supported and securely tied and supported to prevent displacement of the steel during concrete placement. Bar spacing limits and surface condition of reinforcement should conform to the requirements of ACI 318.

3.4.3.1 Concrete cover—The following minimum concrete cover should be provided for reinforcement in cast in place concrete for No. 11 (36) bar, W31 (MW200) or D31 (MD200) wire, and smaller. Cover is measured at the thinnest part of the wall, at the bottom of rustication grooves, or between the raised surfaces of architectural feature panels.

<table>
<thead>
<tr>
<th>Minimum cover, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast against earth</td>
</tr>
<tr>
<td>Cast against forms or mud slabs, or top reinforcement:</td>
</tr>
<tr>
<td>No. 6 (19) bar, W45 (MW290) or D45 (MD290) wire, and larger</td>
</tr>
<tr>
<td>No. 5 (16) bar, W31 (MW200) or D31 (MD200) wire, and smaller</td>
</tr>
<tr>
<td>(b) Concrete support structure:</td>
</tr>
<tr>
<td>Exterior surfaces:</td>
</tr>
<tr>
<td>No. 6 (19) bar, W45 (MW290) or D45 (MD290) wire, and larger</td>
</tr>
<tr>
<td>No. 5 (16) bar, W31 (MW200) or D31 (MD200) wire, and smaller</td>
</tr>
<tr>
<td>Interior surfaces</td>
</tr>
<tr>
<td>Sections designed as beams or columns</td>
</tr>
<tr>
<td>(c) Tank floors and intermediate floor slabs</td>
</tr>
</tbody>
</table>

3.4.3.2 Supports—Supports for reinforcement should conform to the following:

(a) The number of supports should be sufficient to prevent out-of-tolerance deflection of reinforcement, and to prevent overloading any individual support;

(b) Shallow foundation reinforcement placed adjacent to the ground or working slab should be supported by precast concrete block, metal or plastic bar supports;

(c) Reinforcement adjacent to formwork should be supported by metal or plastic bar supports. The portions of bar supports within 1/2 in. (13 mm) of the concrete surface should be noncorrosive or protected against corrosion;

(d) Support wall reinforcement should be provided with plastic supports. Maximum spacing of supports for welded wire fabric should be 5 ft (1.5 m) centers, horizontally and vertically.

3.4.4 Development and splices

3.4.4.1 Development and splice lengths—Development and splices of reinforcement should be in accordance with ACI 318. The location and details of reinforcement development and lap splices should be shown on construction drawings.

3.4.4.2 Welding—Welding of reinforcement should conform to AWS D1.4. A full welded splice should develop 125 percent of the specified yield strength of the bar. Reinforcement should not be tack welded.

3.4.4.3 Mechanical connections—The type, size, and location of any mechanical connections should be shown on construction drawings. A full mechanical connection should develop in tension or compression, as required, 125 percent of the specified yield strength of the bar.

3.5 Concrete finishes

3.5.1 Surface repair

3.5.1.1 Patching materials—Concrete should be patched with a proprietary patching material or site-mixed portland cement mortar. Patching material for exterior surfaces should match the surrounding concrete in color and texture.

3.5.1.2 Repair of defects—Concrete should be repaired as soon as practicable after form removal. Honeycomb and other defective concrete should be removed to sound concrete and patched.

3.5.1.3 Tie holes—Tie holes should be patched, except that manufactured plastic plugs may be used for exterior surfaces.

3.5.2 Formed surfaces—Finishing of formed surfaces should conform to the following:

(a) Exterior exposed surfaces of the support structure and foundations should have a smooth as-cast finish, unless a special formed finish is specified;

(b) Interior exposed surfaces of the support structure should have a smooth as-cast finish;

(c) Concrete not exposed to view may have a rough as-cast finish.

3.5.2.1 Rough as-cast finish—Any form facing material may be used, provided the forms are substantial and sufficiently tight to prevent mortar leakage. The surface is left with the texture imprinted by the form. Defects and tie holes should be patched and fins exceeding 1/4 in. (6 mm) in height should be removed.
3.5.2.2 Smooth as-cast finish—Form facing material and construction should conform to Section 3.3. The surface is left with the texture imprinted by the form. Defects and tie holes should be patched and fins should be removed by chipping or rubbing.

3.5.2.3 Special form finish—A smooth as-cast finish is produced, after which additional finishing is performed. The type of additional finishing required should be specified.

3.5.3 Trowel finishes—Unformed concrete surfaces should be finished in accordance with the following:
- Slabs-on-grade and intermediate floor slabs—steel trowel;
- Dome and flat slab tank floors—floated;
- Foundations—floated;
- Surfaces receiving grout—floated.

3.6—Tolerances

3.6.1 Concrete tolerances—Tolerances for concrete and reinforcement should conform to ACI 117 and the following:
(a) Dimensional tolerances for the concrete support structure:
   - Variation in thickness:
     - wall: -3.0 percent, +5.0 percent
     - dome: -6.0 percent, +10 percent
   - Support wall variation from plumb:
     - in any 5 ft (1.6 m) of height (1/160): 3/8 in. (10 mm)
     - in any 50 ft (16 m) of height (1/400): 1.5 in. (40 mm)
     - maximum in total height: 3 in. (75 mm)
   - Support wall diameter variation: 0.4 percent
     - not to exceed 3 in. (75 mm)
   - Dome tank floor radius variation: 1.0 percent
   - Level alignment variation:
     - from specified elevation: 1 in. (25 mm)
     - from horizontal plane: 1/2 in. (13 mm)
(b) The offset between adjacent pieces of formwork facing material should not exceed the following:
   - Exterior exposed surfaces: 1/8 in. (3 mm)
   - Interior exposed surfaces: 1/4 in. (6 mm)
   - Unexposed surfaces: 1/2 in. (13 mm)
(c) The finish tolerance of troweled surfaces should not exceed the following when measured with a 10 ft (3 m) straightedge or sweep board:
   - Exposed floor slab: 3/16 in. (6 mm)
   - Tank floors: 3/8 in. (20 mm)
   - Concrete support for suspended steel floor tank: 1/4 in. (6 mm)

3.6.2 Out-of-tolerance construction—The effect on the structural capacity of the element should be determined by the responsible design professional if construction does not conform to Section 3.6.1. When structural capacity is not compromised, repair or replacement of the element is not required unless other governing factors, such as lack of fit and aesthetics, require remedial action.

3.7—Foundations

3.7.1 Reinforced Concrete—Concrete, formwork, and reinforcement should conform to the applicable requirements of Chapter 3.

3.7.2 Earthwork

3.7.2.1 Excavations—Foundation excavations should be dry and have stable side slopes. Applicable safety standards and regulations should be followed in constructing excavations.

3.7.2.2 Inspection—Excavations should be inspected prior to concrete construction to ensure that the material encountered reflects the findings of the geotechnical report.

3.7.2.3 Mud mats—A lean concrete mud mat is recommended to protect the bearing stratum, and to provide a working surface for placing reinforcement.

3.7.2.4 Backfill—Backfill should be placed and compacted in uniform horizontal lifts. Fill inside the concrete wall should conform to Section 5.8.2.4. Fill material outside the concrete wall may be unclassified soils free of organic matter and debris. Backfill should be compacted to 90 to 95 percent standard Proctor density (ASTM D 698) or greater.

3.7.2.5 Grading—Site grading around the tank should provide positive drainage away from the tank to prevent ponding of water in the foundation area.

3.7.3 Field inspection of deep foundations—Field inspection by a qualified inspector of foundations and concrete work should conform to the following:
(a) Continuous inspection during pile driving and placement of concrete in deep foundations;
(b) Periodic inspection during construction of drilled piers or piles, during placement of concrete, and upon completion of placement of reinforcement.

3.8—Grout

3.8.1 Steel liner—Unformed steel liner plates that do not match the shape of the concrete floor may be used, provided the liner plate is grouted after welding. The steel liner should be constructed with a 1 in. (25 mm) or larger grout space between the base plate and the concrete member. The space should be completely filled with a flowable grout using a procedure that removes entrapped air. Provide anchorage in areas where the grout pressure is sufficient to lift the plate.

3.8.2 Base plate—A base plate used for the steel bottom configuration should be constructed with a 1 in. (25 mm) or larger grout space between the base plate and the concrete. The space should be completely filled with a non-shrink, non-metallic grout conforming to Section 4.10.5.6. Grout should be placed and achieve required strength before hydrotesting the tank.

CHAPTER 4—DESIGN

4.1—General

4.1.1 Scope—This chapter identifies the minimum requirements for the design and analysis of a concrete-pedestal elevated water tank incorporating a concrete support structure, a steel storage tank, and related elements.

4.1.2 Design of concrete support structure—Analysis and design of the concrete support structure should conform to ACI 318, except as modified here. Design of the concrete support structure elements should conform to Sections 4.8 through 4.10.
4.1.3 Design of steel storage tank—The materials, design, fabrication, erection, testing, and inspection of the steel storage tank should conform to recognized national standards.

4.1.4—Design of other elements

4.1.4.1 Concrete members—Design of concrete members such as foundations, floor slabs, and similar structural members should conform to ACI 318, and the requirements of Sections 4.11 and 5.8.

4.1.4.2 Non-concrete members—Design of non-concrete related elements such as appurtenances, accessories and structural steel framing members should conform to recognized national standards for the type of construction.

4.1.4.3 Safety related components—Handrails, ladders, platforms, and similar safety related components should conform to the applicable building code, and to Occupational Safety and Health Administration standards.

4.1.5 Unit weight—The unit weight of materials used in the design for the determination of gravity loads should be as follows, except where materials are known to differ or specifications require other values:

(a) Reinforced concrete: 150 lb/ft³ (2400 kg/m³);
(b) Soil backfill: 100 lb/ft³ (1600 kg/m³);
(c) Water: 62.4 lb/ft³ (1000 kg/m³);
(d) Steel: 490 lb/ft³ (7850 kg/m³);

4.2—Loads

4.2.1 General—The structure should be designed for loads not less than those required for an ASCE 7 Category IV structure, or by the applicable building code.

4.2.2 Structural loads—The loads in Section 4.2.2.1 through 4.2.2.8 should be considered to act on the structure as a whole.

4.2.2.1 Dead loads—The weight (mass) of structural components and permanent equipment.

4.2.2.2 Water load—The load produced by varying water levels ranging from empty to overflow level.

4.2.2.3 Live loads—Distributed and concentrated live loads acting on the tank roof, access areas, elevated platforms, intermediate floors or equipment floors. The distributed roof live load should be the greater of snow load determined in Section 4.5, or 15 lb/ft² (0.72 kPa) times the horizontal projection of the roof surface area to the eave line. Unbalanced loading should be considered in the design of the roof and its supporting members.

4.2.2.4 Environmental loads—Environmental loads should conform to:

(a) Snow loads: Section 4.5;
(b) Wind forces: Section 4.6;
(c) Seismic forces: Section 4.7.

4.2.2.5 Vertical load eccentricity—Eccentricity of dead and water loads that cause additional overturning moments to the structure as a whole should be accounted for in the design. The additional overturning moment is the dead and water load times the eccentricity $e_g$, which should not be taken as less than

$$e_g = e_o + \frac{e}{400}$$

(4-1a)

The minimum vertical load eccentricity $e_o$ is 1 in. (25 mm). Where tilting of the structure due to non-uniform settlement is estimated to exceed 1/800, the eccentricity $e_g$ should not be taken as less than

$$e_g = e_o + \frac{e}{800} + \tan \theta_s$$

(4-1b)

4.2.2.6 Construction loads—Temporary loads resulting from construction activity should be considered in the design of structural components required to support construction loads.

4.2.2.7 Creep, shrinkage, and temperature—The effects of creep, shrinkage, and temperature effects should be considered. ACI 209R provides guidance for these conditions.

4.2.2.8 Future construction—Where future construction, such as the addition of intermediate floors is anticipated, the load effects should be included in the original design. Future construction dead and live loads should be included in the Group 1 load combinations. Only that portion of the dead load $D$ existing at the time of original construction should be included in the Group 2 load combinations.

4.2.3 Factored load combinations—Load factors and load combinations for the Strength Design Method should conform to the following. The load terms are as defined in Section 1.6.1.

4.2.3.1 Group 1 load combinations—Where the structural effects of applied loads are cumulative the required strength should not be less than:

Load Combination:

\[
U1.1 \quad 1.4D + 1.6F \\
U1.2 \quad 1.4(D + G) + 1.6F + 1.7(S + L) \\
U1.3 \quad 1.1(D + G) + 1.2F + 1.3(L + W) \\
U1.4 \quad \gamma_e \left[1.2(D + F) + 0.5(G + L + E) + E_v \right] \\
\]

4.2.3.2 Group 2 load combinations—Where $D$, $L$, or $F$ reduce the effect of $W$ or $E$, as in uplift produced by overturning moment, the required strength should not be less than:

Load Combination:

\[
U2.1 \quad 0.9D + 1.3W \\
U2.2 \quad \gamma_e \left[0.9(D + F) + E\right] + E_v \\
\]

4.2.3.3 Differential settlement, creep, shrinkage, and temperature—Where structural effects of differential settlement, creep, shrinkage or temperature effects are significant: 1.47 should be included with Load Combinations U1.1 and U1.2, and 1.17 should be included with Load Combinations U1.3 and U1.4. Where structural effects $T$ are significant: 1.17 should be included with Group 2 loads when $T$ is additive to $W$ or $E$.

4.2.3.4 Vertical seismic load effect—The vertical seismic load effect $E_v$ in Eq. U1.4 and U2.2 should conform to the requirements of the project documents, or the applicable building code. Where ASCE 7 is specified, $E_v$ is $0.5C_a(D + F)$.

4.2.3.5 Partial seismic load factor—The partial seismic load factor $\gamma_e$ should conform to the requirements of the project documents, or the applicable building code. Where ASCE 7 is specified, $\gamma_e$ is 1.1 for concrete elements.
4.2.4 Unfactored load combinations—Unfactored service load combinations should conform to the following. The load terms are as defined in Section 1.6.1.

4.2.4.1 Group 1 load combinations—Where the structural effects of applied loads are cumulative the unfactored service load combination should not be less than:

- Load Combination:
  - S1.1 \( D + F \)
  - S1.2 \( D + F + G + S + L \)
  - S1.3 \( 0.75(D + F + G + L + W) \)
  - S1.4 \( 0.75(D + F + G + L + E1 + E_v) \)

4.2.4.2 Group 2 load combinations—Where \( D, L, \) or \( F \) reduce the effect of \( W \) or \( E \), as in uplift produced by overturning moment, the required strength should not be less than:

- Load Combination:
  - S2.1 \( 0.75(D + W) \)
  - S2.2 \( 0.75(D + F + E) + E_v \)

4.2.4.3 Differential settlement, creep, shrinkage, and temperature—Where structural effects of differential settlement, creep, shrinkage or temperature effects are significant:

- Structural effects \( T \) are significant:
  - 0.75\( T \) should be included with Load Combinations S1.1 and S1.2, and 0.75\( T \) should be included with Load Combinations S1.3 and S1.4. Where structural effects \( T \) are significant:
  - 0.75\( T \) should be included with Group 2 loads when \( T \) is additive to \( W \) or \( E \).

4.2.4.4 Vertical seismic load effect—The vertical seismic load effect \( E_v \) in Eq. S1.4 and S2.2 should conform to the requirements of the project documents, or the applicable building code. Where ASCE 7 is specified, \( E_v \) is 0.75 \( 0.5C_a \) \( (D + F) \).

4.3—Strength requirements

4.3.1 General—Concrete portions of the structure should be designed to resist the applied loads that may act on the structure and should conform to this document.

4.3.1.1 Specified concrete strength—Specified compressive strength \( f'_c \) of concrete components should conform to Section 3.2.2.2 and applicable sections of Chapter 4.

- The specified yield strength of reinforcement \( f_y \) should not exceed 80,000 psi (550 MPa).

4.3.2—Design methods

4.3.2.1 Strength design method—Structural concrete members should be proportioned for adequate strength in accordance with the Strength Design provisions of ACI 318 and this document. Loads should not be less than the factored loads and forces in Section 4.2.3. Strength reduction factors \( \phi \) should conform to ACI 318 and to applicable sections of Chapter 4.

4.3.2.2 Alternate design method—The Alternate Design Method of ACI 318 is an acceptable method for design. Unfactored load combinations should conform to Section 4.2.4.

4.3.3—Minimum reinforcement

4.3.3.1 Flexural members—Where flexural reinforcement is required by analysis in the support structure and foundations supported by piling and drilled piers, the minimum reinforcement ratio \( p \) should not be less than \( 3\sqrt{f'_c / f_y} \) nor 200/\( f_y \) in in.-lb units \( (0.25\sqrt{f'_c / f_y} \) nor 1.4/\( f_y \) in SI units).

A smaller amount of reinforcement may be used if at every section the area of tensile reinforcement provided is at least one-third greater than that required by analysis.

4.3.3.2 Direct tension members—In regions of significant direct tension the minimum reinforcement ratio \( p_g \) should not be less than 5\( \sqrt{f'_c / f_y} \) in in.-lb units \( (0.42\sqrt{f'_c / f_y} \) in SI units). A smaller amount of reinforcement may be used if the area of tensile reinforcement provided is at least one-third greater than that required by analysis.

4.4—Serviceability requirements

4.4.1 General—Concrete portions of the structure should conform to this document to ensure adequate performance at service loads. The following should be considered.

- (a) Deflection of flexural beam or slab elements should conform to ACI 318.

- (b) Control of cracking should conform to Section 4.4.2 and applicable sections of Chapter 4.

- (c) Settlement of foundations should conform to Sections 4.12.3 and 4.12.5.

4.4.2 Control of cracking—Cracking and control of cracking should be considered at locations where analysis indicates flexural tension or direct tension stresses occur.

- Where control of cracking is required, sections should be proportioned such that quantity \( z_s \) does not exceed 145 kips per inch \((25,400 \text{ N/mm})\) for sections subjected to flexure, or 130 kips per inch \((22,800 \text{ N/mm})\) for sections subjected to direct tension. The quantity \( z_s \) is determined by:

\[
z_s = f'_s \sqrt{d_c A}
\]  

(4-2)

- Calculated stress in reinforcement \( f'_s \) is for Load Combination S1.1 in Section 4.2.4.1. Alternatively, \( f'_s \) may be taken as 60 percent of the specified yield strength \( f_y \). The clear cover used in calculating the distance from the extreme tension fiber to the tension steel centroid \( d_c \) should not exceed 2 in. \((50 \text{ mm})\) even though the actual cover is larger.

4.5—Snow Loads

4.5.1—General

4.5.1.1 Scope—This section covers determination of minimum snow loads for design and is based on ASCE 7 for Category IV structures. Larger loads should be used where required by the applicable building code.

4.5.1.2 Definitions—Certain terms used in this section are defined as follows:

- Crown—highest point of the roof at centerline of tank.
- Eaves—highest level at which the tank diameter is maximum; or the 70-deg point of the roof slope of curved or conical roofs, if present. The 70-deg point is the radius at which the roof slope is 70 deg measured from the horizontal.
- Cone roof—monoslope roof having a constant slope from crown to eaves.
- Conical roof—a cone roof combined with an edge cone or a doubly curved edge segment.
- Curved roof—dome, ellipsoidal, or other continuous shell roofs with increasing slope from crown to eaves; or the doubly curved portion of a conical roof.
Roof slope $\theta_r$—roof slope at a point measured from the horizontal.

Effective curved roof slope $\theta_c$—slope of a straight line from the eaves (or the 70-deg point if present) to the crown of a curved roof, or a conical roof.

4.5.1.3 Limitations—The provisions of Section 4.5 are applicable to cone, conical, and curved roofs concave downward without steps or abrupt changes in elevation.

4.5.2 Roof snow load—The unfactored snow load acting on the structure is the sum of the uniformly distributed snow load acting on any portion of a roof times the horizontal projected area on which $w_s$ acts. The uniformly distributed snow load $w_s$ is the larger value determined in Sections 4.5.2.1 and 4.5.2.2.

4.5.2.1 Sloped roof snow load—Portions of a roof having a slope $\theta_r$ exceeding 70 deg should be considered free of snow load. Where roof slope $\theta_r$ is 70 deg or less, the distributed snow load is given by

$$w_s = 0.76 \, C_r \, I \, p_g$$

(4-3a)

The ground snow load $p_g$ is in accordance with Section 4.5.2.3, and the roof slope factor $C_r$ is in accordance with Section 4.5.2.4. The snow importance factor $I$ is 1.2.

4.5.2.2 Minimum snow load—The minimum snow load acting on cone roofs with slope $\theta_c$ less than 15 deg and curved roofs with slope $\theta_c$ less than 10 deg is the larger value determined from Eq. (4-3b) and (4-3c) when the ground snow load $p_g$ is greater than zero

$$w_s = C_r \, p_{20} \, I \quad \text{for } p_g > p_{20}$$

(4-3b)

$$w_s = C_r \, (I \, p_g + p_r) \quad \text{for } p_g \leq p_{20}$$

(4-3c)

where $p_{20} = 20 \, \text{lb/ft}^2 \quad (0.96 \, \text{kPa})$ ground snow load

The rain-snow surcharge $p_r$ is 5 lb/ft$^2$ (0.24 kPa). For roof slopes steeper than 1 vertical to 24 horizontal (greater than 2.38 deg from the horizontal) it may be reduced by 0.24$I$ $p_g$ up to a maximum reduction of 5 lb/ft$^2$ (0.24 kPa).

4.5.2.3 Ground snow load—The ground snow load $p_g$ should be based on an extreme-value statistical analysis of weather records using a 2 percent annual probability of being exceeded (50-year mean recurrence interval). In the contiguous United States and Alaska ground snow load $p_g$ should be determined from Fig. 7-1 or Table 7-1 in ASCE 7.

4.5.2.4 Roof slope factor—The roof slope factor at any point on the roof is given by:

$$C_r = 1.27 - \frac{\theta_r}{55}, \text{not greater than 1.0 nor less than zero. For curved roofs or portions of roofs that are curved the distribution of snow load should be assumed to vary linearly between points at 15 and 30 deg, and the eaves. Linear interpolation should be used where the roof slope at the eaves is less than 70 deg.}$$

4.6—Wind forces

4.6.1 Scope—This section covers determination of minimum service load wind forces for design, and is based on

ASCE 7 for Category IV structures. Larger loads should be used where required by the applicable building code.

4.6.2—Wind speed

4.6.2.1 Basic wind speed—The basic wind speed $V_b$ is the 3-sec gust speed at 33 ft (10 m) above ground for Exposure C category, and is associated with a 2 percent annual probability of being exceeded (50-yr mean recurrence interval). In the contiguous United States and Alaska basic wind speed $V_b$ may be determined from Fig. 6-1 in ASCE 7.

4.6.2.2 Wind speed-up—Wind speed-up over hills and escarpments should be considered for structures sited on the upper half of hills and ridges or near the edge of escarpments.

4.6.3 Design wind force—The service load wind force $W$ acting on the structure is the sum of the forces calculated from Section 4.6.3.1.

4.6.3.1—The design wind force $F_w$ acting on tributary area $A_f$ is

$$F_w = C_f \, p_z \, A_f$$

(4-4)

where

$$C_f = \text{wind force drag coefficient}$$

$= 0.6$, for cylindrical surfaces

$= 0.5$, for double curved surfaces or cones with an apex angle > 30 deg.

The wind pressure $p_z$ at height $z$ above ground level is in accordance with Section 4.6.3.2.

4.6.3.2—Wind pressure $p_z$ is

$$p_z = C_e \, q_s \, I \quad \text{not less than 30 lb/ft}^2 \quad (1.44 \, \text{kPa})$$

(4-5)

where

$$q_s = 0.00256 (V_b)^2, \text{lb/ft}^2; \text{wind stagnation pressure}$$

$$q_s = 0.000613 (V_b)^2, \text{kPa}; \text{wind stagnation pressure in SI units}$$

The basic wind speed $V_b$ is in accordance with Section 4.6.2.1, and the combined height and gust response factor $C_e$ is in accordance with Table 4.6.3(a). The wind importance factor $I$ is 1.15.

4.6.3.3 Exposure category—The wind exposure in which the structure is sited should be assessed as being one of the following:

(a) Exposure B: urban and suburban areas. Characterized by numerous closely spaced obstructions having the size of single-family dwellings or larger. This exposure is limited to areas where the terrain extends in all directions a distance of 1500 ft (460 m) or 10 times the structure height, whichever is greater;

(b) Exposure C: flat and generally open terrain, with scattered obstructions having heights generally less than 30 ft (9 m);

(c) Exposure D: flat, unobstructed areas exposed to wind flowing over open water for a distance of at least one mile (1600 m). This exposure extends inland from the shoreline a distance of 1500 ft (460 m) or 10 times the structure height, whichever is greater.
Table 4.6.3—Combined height and gust factor: \( C_e \)

<table>
<thead>
<tr>
<th>Height above ground level, ft (m)</th>
<th>Exposure B</th>
<th>Exposure C</th>
<th>Exposure D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 75 (23)</td>
<td>0.73</td>
<td>1.01</td>
<td>1.16</td>
</tr>
<tr>
<td>100 (30)</td>
<td>0.79</td>
<td>1.07</td>
<td>1.22</td>
</tr>
<tr>
<td>150 (46)</td>
<td>0.89</td>
<td>1.17</td>
<td>1.31</td>
</tr>
<tr>
<td>200 (61)</td>
<td>0.96</td>
<td>1.24</td>
<td>1.37</td>
</tr>
<tr>
<td>250 (76)</td>
<td>1.03</td>
<td>1.30</td>
<td>1.43</td>
</tr>
<tr>
<td>300 (91)</td>
<td>1.08</td>
<td>1.36</td>
<td>1.47</td>
</tr>
</tbody>
</table>

**4.7—Seismic forces**

**4.7.1—General**

**4.7.1.1 Scope**—This section covers determination of minimum factored seismic forces for design, and is based on ASCE 7 Category IV structures. Larger loads should be used where required by the applicable building code.

**4.7.1.2 Definitions**—Certain terms used in this section are defined as follows:

- **Base**—The level at which the earthquake motions are considered to be imparted to the structure.
- **Base Shear** \( V \)—The total design lateral force or shear at base of structure.
- **Gravity load** \( W_G \)—Dead load and applicable portions of other loads defined in Section 4.7.6.3 that is subjected to seismic acceleration.

**4.7.1.3 Limitations**—The provisions of Section 4.7 are applicable to sites where the effective peak ground acceleration coefficient \( A_v \) is 0.4 or less.

**4.7.2 Design seismic force**—The factored design seismic forces acting on the structure should be determined by one of the following procedures. Structures should be designed for seismic forces acting in any horizontal direction.

**4.7.2.1 Equivalent lateral force procedure**—The equivalent lateral force procedure of Section 4.7.6 may be used for all structures.

**4.7.2.2 Alternative procedures**—Alternative lateral force procedures, using rational analysis based on well established principles of mechanics, may be used in lieu of the equivalent lateral force procedure. Base shear \( V \) used in design should not be less than 70 percent of that determined by Section 4.7.6.

**4.7.2.3**—Seismic analysis is not required where the effective peak velocity-related acceleration coefficient \( A_v \) is less than 0.05.

**4.7.3 Soil profile type**—Where the peak effective velocity-related ground acceleration \( A_v \) is 0.05 or greater, the soil profile type should be classified in accordance with Table 4.7.3 by a qualified design professional using the classification procedure given in ASCE 7.

**4.7.4—Seismic coefficients**

**4.7.4.1 Effective peak ground acceleration coefficients**—The effective peak acceleration \( A_d \) and effective peak velocity-related acceleration coefficient \( A_v \) should be determined from Maps 9-1 and 9-2, respectively, of ASCE 7. Where site-specific ground motions are used or required, they should be developed on the same basis, with 90 percent probability of not being exceeded in 50 years.

**4.7.4.2 Seismic acceleration coefficients**—Seismic acceleration coefficients \( C_a \) and \( C_v \) should be determined from Table 4.7.4.

At sites with soil profile F, seismic coefficients should be determined by site specific geotechnical investigation and dynamic site response analyses.

**4.7.4.3 Response modification coefficient**—The response modification coefficient \( R \) used in design should not exceed 2.0.

**4.7.5—Structure period**

**4.7.5.1 Fundamental period**—The fundamental period of vibration \( T \) of the structure should be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis.

**4.7.5.2 Single lumped-mass approximation**—The structure period \( T \) may be calculated from Eq. (4-6) when the water load is 80 percent or more of the total gravity load \( W_G \)

\[
T = 2\pi \sqrt{\frac{W_G}{g k_c}}
\]

The single lumped-mass structure weight \( W_L \) consists of:
- (a) Self-weight of the tank and tank floor;
- (b) Maximum of two-thirds the self-weight of concrete support wall; and
- (c) Water load.

**4.7.6—Equivalent lateral force procedure**

**4.7.6.1 Seismic base shear**—The total seismic shear \( V \) in a given direction is determined by

\[
V = C_s W_G
\]

The seismic response coefficient \( C_s \) is in accordance with Section 4.7.6.2, and the gravity load \( W_G \) is in accordance with Section 4.7.6.3.
4.7.6.2 Seismic response coefficient—The seismic response coefficient $C_s$ is the smaller value determined from Eq. (4-8a) and (4-8b)

$$C_s = \frac{1.2C_v}{RT^{2/3}}$$  \hspace{1cm} (4-8a)

$$C_s = \frac{2.5C_a}{R}$$ \hspace{1cm} (4-8b)

The minimum value of $C_s$ should not be less than

$$C_s = 0.5C_a$$ \hspace{1cm} (4-9)

4.7.6.3 Gravity load—The gravity load $W_G$ includes: the total dead load above the base, water load, and a minimum of 25 percent of the floor live load in areas used for storage.

4.7.7 Force distribution—The total lateral seismic force $V$ should be distributed over the height of the structure in proportion to the structure weight by Eq. (4-10a) when the dead load is less than approximately 25 percent of the total weight. Where the dead is greater the distribution of lateral seismic force should be determined Eq. (4-10b)

$$F_x = V \frac{W_x}{\sum_{i=1}^{n} W_i}$$ \hspace{1cm} (4-10a)

$$F_x = V \frac{W_x}{\sum_{i=1}^{n} (W_i)^{k}}$$ \hspace{1cm} (4-10b)

The exponent $k$ is 1.0 for a structure period less than 0.5 sec, and 2.0 for a structure period of 2.5 sec. Interpolation may be used for intermediate values, or $k$ may be taken as 2.0 for structure periods greater than 0.5 sec.

4.7.8 Lateral seismic shear—The lateral seismic shear $V_x$ acting at any level of the structure is determined by

$$V_x = \sum_{i=1}^{x} F_i$$ \hspace{1cm} (4-11)

where $\Sigma F_i$ is from the top of the structure to the level under consideration.

4.7.9—Overturning moment

4.7.9.1—The overturning moment at the base $M_o$ is determined by

$$M_o = \sum_{i=1}^{n} (F_i l_i)$$ \hspace{1cm} (4-12)

4.7.9.2—The overturning moment $M_x$ acting at any level of the structure is the larger value determined from Eq. (4-13a) and (4-13b)

$$M_x = \sum_{i=1}^{x} F_i (l_i - l_x)$$ \hspace{1cm} (4-13a)

$$M_x = M_o \left(1 - 0.5 \frac{l_x}{l_{cg}}\right)$$ \hspace{1cm} (4-13b)

4.7.10—Other effects

4.7.10.1 Torsion—The design should include an accidental torsional moment caused by an assumed displacement of the mass from its actual location by a distance equal to 5 percent of the support wall diameter. Torsional effects may be ignored when the torsional shear stress is less than 5 percent of the shear strength determined in Section 4.8.6.8.

4.7.10.2 P-delta effects—P-delta effects may be ignored when the increase in moment is less than 10 percent of the moment without P-delta effects.

4.7.10.3 Steel tank anchorage—The anchorage of the steel tank to the concrete support should be designed for twice the design seismic force determined in accordance with Section 4.7.2, at the level of the anchorage.

4.8—Support wall

4.8.1 General—Design of the concrete support wall should be in accordance with ACI 318 except as modified in this document. Other methods of design and analysis may be used. The minimum wall reinforcement should not be less than required by Table 4.8.2. Portions of the wall subjected to significant flexure or direct tension loads should conform to Sections 4.3.3 and 4.4.2.

4.8.2—Details of wall and reinforcement
4.8.2.1 Minimum wall thickness—Wall thickness \( h \) should not be less than 8 in. (200 mm). The thickness \( h \) is the structural thickness, exclusive of any rustications, fluting or other architectural relief.

4.8.2.2 Specified compressive strength—The specified compressive strength of concrete should not be less than required in Section 3.2.2.2 nor greater than 6000 psi (41 MPa).

4.8.2.3 Reinforcement—Wall reinforcement should conform to Table 4.8.2. Not more than 60 percent nor less than 50 percent of the minimum reinforcement in each direction specified in Table 4.8.2 should be distributed to the exterior face, and the remainder to the interior face.

4.8.2.4 Concrete cover—Concrete cover to reinforcement should conform to Section 3.4.3.1.

4.8.2.5 Transverse reinforcement—Cross ties are required in walls at locations where:

(a) Vertical reinforcement is required as compression reinforcement and the reinforcement ratio \( p_g \) is 0.01 or more;

(b) Concentrated plastic hinging or inelastic behavior is expected during seismic loading.

Where cross ties are required, the size and spacing should conform to ACI 318 Section 7.10, and Section 21.4.4 in seismic areas.

4.8.3—Vertical load capacity

4.8.3.1 Design load—The factored axial wall load per unit of circumference \( P_{nw} \) should conform to Section 4.2.3.

4.8.3.2 Axial load strength—Design for vertical load capacity per unit length of circumference should be based on

\[
P_{nw} \leq \phi P_{nw}
\]

where \( \phi = 0.7 \).

The nominal axial load strength per unit length of circumference \( P_{nw} \) should not exceed

\[
P_{nw} = \beta_w C_w f'_c A_w
\]

The wall slenderness coefficient \( \beta_w \) is 0.55.

4.8.3.3 Other methods—\( C_w \) and \( \beta_w \) may be determined by other design methods, subject to the limitations of Section 4.8.1. Other methods should consider:

(a) The magnitude of actual, as-built, deviations from the theoretical geometry;

(b) The effect on the wall stresses of any surface relief, or other patterning that may be incorporated into the wall concrete;

(c) Creep and shrinkage of concrete;

(d) Inelastic material properties;

(e) Cracking of concrete;

(f) Location, amount, and orientation of reinforcing steel;

(g) Local effects of stress raisers (for example, doorways and pilasters);

(h) Possible deformation of supporting elements, including foundation settlements;

(i) Proximity of the section being designed to beneficial influences, such as restraint by foundation or tank floor.

4.8.3.4 Foundation rotation—Bending in the support wall due to radial rotation of the foundation should be included in the support wall design, if applicable.

4.8.4—Circumferential bending

4.8.4.1—Horizontal reinforcement should be provided in each face for circumferential moments arising from ovaling of the wall due to variations in wind pressures around the wall circumference. The factored design wind ovaling moment should be determined by multiplying \( M_h \) by the wind load factor defined in Section 4.2.3.

4.8.4.2—At horizontal sections through the wall that are remote from a level of effective restraint where circularity is maintained, the service load wind ovaling moment per unit of height \( M_h \) may be determined from

\[
M_h = 0.052 p_z d_w^2 \]  

where \( p_z \) is calculated in accordance with Section 4.6.3.2. Other means of analysis may be used.

---

### Table 4.8.2—Minimum wall reinforcement requirements

<table>
<thead>
<tr>
<th>Reinforcement parameter</th>
<th>Seismic coefficient ( A_1 &lt; 0.20 )</th>
<th>Seismic coefficient ( A_1 \cdot 0.20 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum reinforcement ratio ( \beta )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertically—No. 11 (36) bar and smaller</td>
<td>0.0015</td>
<td>0.0025</td>
</tr>
<tr>
<td>Horizontally—No. 5 (16) bar and smaller</td>
<td>0.0020</td>
<td>0.0025</td>
</tr>
<tr>
<td>—No. 6 (19) bar and larger</td>
<td>0.0025</td>
<td>0.0030</td>
</tr>
<tr>
<td>Type of reinforcement permitted</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deformed bars</td>
<td>ASTM A 615 / A 615M or A 706 / A 706M</td>
<td>ASTM A 615 / A 615M or A 706 / A 706M</td>
</tr>
<tr>
<td>Plain or deformed</td>
<td>ASTM A 185 or A 497</td>
<td></td>
</tr>
<tr>
<td>Maximum specified yield strength ( f_y ) permitted</td>
<td>60,000 psi (420 MPa)</td>
<td>60,000 psi (420 MPa)</td>
</tr>
</tbody>
</table>

\( \beta \) Minimum reinforcement ratio applies to the gross concrete area.

\( \beta \) Mill tests demonstrating conformance to ACI 318 are required when ASTM A 615 / A 615M bars are used for reinforcement resisting earthquake-induced flexural and axial forces. ASTM A 615 / A 615M, ASTM A 185, and ASTM A 497 are permitted for reinforcement resisting other forces, and for shrinkage and temperature steel.
4.8.4.3—The wind ovalling moment \( M_h \) may be considered to vary linearly from zero at a diaphragm elevation to the full value at a distance \( 0.5 \, d_w \) from the diaphragm.

4.8.5—Openings in walls

4.8.5.1—The effects of openings in the wall should be considered in the design. Wall penetrations having a horizontal dimension of 3 ft \((0.9 \, \text{m})\) or less and a height of 12 ft \((3.6 \, \text{m})\) or less may be designed in accordance with Section 4.8.5.2. Otherwise, the design should conform to Sections 4.8.5.3 through 4.8.5.5.

4.8.5.2 Simplified method—Where detailed analysis is not required, minimum reinforcement around the opening is the larger amount determined by:

(a) Vertical and horizontal reinforcement interrupted by the opening should be replaced by reinforcement having an area not less than 120 percent of the interrupted reinforcement, half placed each side of the opening, and extending past the opening a distance not less than half the transverse opening dimension;

(b) An area each side of the opening equal to \( 0.75 b_d \) should be evaluated for vertical load capacity, and reinforced as required. The load acting on this area should be half the vertical force interrupted by the opening plus the average vertical load in the wall at mid-height of the opening;

(c) The effective unsupported column length \( k l \) should not be less than 0.85\( b_d \);

(d) The effective columns should be analyzed by the slender column procedures of ACI 318 and reinforced accordingly with bars on the inside and outside faces of the wall. Transverse reinforcement should conform to ACI 318 Section 7.10, and Section 21.4.4 in seismic areas;

(e) The effective column should be checked for the effects of vehicle impact if the opening is to be used as a vehicle entrance through the support wall.

4.8.5.3 Effective column—The wall adjacent to an opening should be designed as a braced column in accordance with ACI 318 and the following:

(a) Each side of the opening should be designed as a reinforced concrete column having an effective width equal to the smaller of \( 5h \), 6 ft \((1.8 \, \text{m})\), or \( 0.5b_d \);

(b) The effective column should be designed to carry half the vertical force interrupted by the opening plus the average vertical load in the wall at mid-height of the opening;

(c) The effective unsupported column length \( k l \) should not be less than 0.85\( b_d \);

(d) The effective columns should be analyzed by the slender column procedures of ACI 318 and reinforced accordingly with bars on the inside and outside faces of the wall. Transverse reinforcement should conform to ACI 318 Section 7.10, and Section 21.4.4 in seismic areas;

(e) The effective column should be checked for the effects of vehicle impact if the opening is to be used as a vehicle entrance through the support wall.

4.8.5.4 Pilasters—Monolithic pilasters may be used adjacent to openings. Such pilasters should extend above and below the opening a sufficient distance to effect a smooth transition of forces into the wall without creating excessive local stress concentrations. The transition zone where pilasters are terminated should be thoroughly analyzed and additional reinforcement added if required for local stresses. The reinforcement ratio \( p_p \) should not be less than 0.01.

4.8.5.5 Horizontal reinforcement—Additional horizontal reinforcement should be provided above and below openings in accordance with Eq. (4-18), and should be distributed over a height not exceeding \( 3h \)

\[
A_s = \frac{0.14 P_{uw} b_d}{f_y} \tag{4-18}
\]

where \( \phi = 0.9 \). \( P_{uw} \) applies at the level of the reinforcement being designed. The quantity \( P_{uw} b_d \) is expressed in lb (N). The reinforcement yield strength \( f_y \) used in Eq. (4-18) should not exceed 60,000 psi (420 MPa).

4.8.5.6 Development of reinforcement—Additional reinforcement at openings is to be fully developed beyond the opening in accordance with ACI 318. Additional horizontal reinforcement should project at least half a development length beyond the effective column or pilaster width of Sections 4.8.5.3 or 4.8.5.4.

4.8.5.7 Local effects below openings—Where the combined height of wall and foundation below the opening is less than one-half the opening width the design should conform to Section 4.11.6.6.

4.8.6—Shear design

4.8.6.1 Radial shear—Design of the concrete support wall for radial shear forces should conform to Chapter 11 of ACI 318.

4.8.6.2 In-plane shear—Design of the concrete support wall for in-plane shear forces caused by wind or seismic forces should conform to the requirements of Sections 4.8.6.3 through 4.8.6.10.

4.8.6.3 Design forces—The shear force \( V_u \) and simultaneous factored moment \( M_u \) should be obtained from the lateral load analysis for wind and seismic forces.

4.8.6.4 Shear force distribution—The shear force distribution in the concrete support wall should be determined by a method of analysis that accounts for the applied loads and structure geometry. The simplified procedure of Section 4.8.6.5 may be used when the ratio of openings to effective shear wall width \( \psi \) does not exceed 0.5.

4.8.6.5 Shear force—The shear force \( V_u \) may be considered to be resisted by two equivalent shear walls parallel to the direction of the applied load. The length of each shear wall should not exceed 0.78\( d_w \). The shear force \( V_{uw} \) acting on an equivalent shear wall should not be less than:

(a) In sections of the wall without openings or sections with openings symmetric about the centerline the factored shear force \( V_{uw} \) assigned to each shear wall is

\[
V_{uw} = 0.5 \, V_u \tag{4-19}
\]

(b) In sections of the wall with openings not symmetrical about the centerline

\[
V_{uw} = 0.5V_u \left( 1 + \frac{\psi}{2 - \psi} \right) \tag{4-20}
\]

where\[\psi = \frac{b_x}{0.78d_w}\]

\( b_x \) is the cumulative width of openings in the effective shear wall width 0.78\( d_w \). The dimensions \( b_x \) and \( d_w \) are expressed in in. (mm).
4.8.6.6 Shear area — The effective horizontal concrete wall area \( A_{cv} \) resisting the shear force \( V_{uw} \) should not be greater than

\[
A_{cv} = 0.78 (1 - \psi) d_w h \tag{4-21}
\]

where the dimensions of \( d_w \) and \( h \) are expressed in in. (mm).

4.8.6.7 Maximum shear — The distributed shear \( V_{uw} \) should not exceed:

(a) \( 8\sqrt[6]{f_c} A_{cv} \) in in.-lb units \([12/3]\sqrt[6]{f_c} A_{cv} \) in SI units \( \) when Eq. (4-19) controls, and

(b) \( 10\sqrt[6]{f_c} A_{cv} \) in in.-lb units \([5/6]\sqrt[6]{f_c} A_{cv} \) in SI units \( \) when Eq. (4-20) controls.

4.8.6.8 Shear strength — Design for in-plane shear should be based on

\[
V_{uw} \leq \Phi V_n \tag{4-22}
\]

where \( \psi = 0.85 \). The nominal shear strength \( V_n \) should not exceed the shear force calculated from

\[
V_n = (\alpha_c \sqrt[6]{f_c} + \rho_h f_y) A_{cv} \tag{4-23}
\]

where

\[
\alpha_c = 6 - \frac{2.5 M_u}{V_u d_w} \text{ but not less than 2.0 nor greater than 3.0; in.-lb units.}
\]

\[
\alpha_c = 0.5 - \frac{0.21 M_u}{V_u d_w} \text{ but not less than 1/6 nor greater than 1/4; SI units.}
\]

\( M_u \) and \( V_u \) are the total factored moment and shear occurring simultaneously at the section under consideration, and \( \rho_h \) is the ratio of horizontal distributed shear reinforcement on an area perpendicular to \( A_{cv} \).

4.8.6.9 Design location — The nominal shear strength \( V_n \) should be determined at a distance above the foundation equal to the smaller of 0.39 \( d_w \) or the distance from the foundation to mid-height of the largest opening, or set of openings with the largest combined \( \psi \).

4.8.6.10 Reinforcement — Minimum reinforcement should conform to Table 4.8.2. In regions of high seismic risk, reinforcement should also conform to the following:

(a) When \( V_{uw} \) exceeds \( 8\sqrt[6]{f_c} A_{cv} \) in in.-lb units \([12/3]\sqrt[6]{f_c} A_{cv} \) in SI units \( \) the minimum horizontal and vertical reinforcement ratios should not be less than 0.0025.

(b) When \( V_{uw} \) exceeds \( 2\sqrt[6]{f_c} A_{cv} \) in in.-lb units \([5/6]\sqrt[6]{f_c} A_{cv} \) in SI units \( \) two layers of reinforcement should be provided.

(c) Where shear reinforcement is required for strength, the vertical reinforcement ratio \( \rho_v \) should not be less than the horizontal reinforcement ratio \( \rho_h \).

4.9 — Tank floors

4.9.1 — General

4.9.1.1 Scope — This section covers design of concrete flat slab and dome floors of uniform thickness used as tank floors, and suspended steel floors. Section 4.10 discusses the interaction effects of the concrete support structure and the storage tank that should be considered in the design.

4.9.1.2 Loads — The loads and load combinations should conform to Sections 4.2.3 and 4.2.4. Loads acting on the tank floor are distributed dead and water loads, and concentrated loads from the access tube, piping and other supports.

4.9.2 — Flat slab floors

4.9.2.1 Design — Concrete slab floors should be designed in accordance with ACI 318, except as modified here. Specified compressive strength of concrete \( f'_c \) should not be less than required in Section 3.2.2.2.

4.9.2.2 Slab stiffness — The stiffness of the slab should be sufficient to prevent rotation under dead and water loads that could cause excessive deformation of the attached wall and steel tank elements. The stiffness of the slab should be calculated using the gross concrete area, and one-half the modulus of elasticity of concrete.

4.9.2.3 Minimum reinforcement — Reinforcement should not be less than 0.002 times the gross concrete area in each direction. Where tensile reinforcement is required by analysis, the minimum reinforcement should conform to Section 4.3.3.

4.9.2.4 Crack control — Distribution of tension reinforcement required by analysis should conform to Section 4.4.2.

4.9.3 — Dome floors

4.9.3.1 Design — Concrete dome floors should be designed on the basis of elastic shell analysis. Consideration of edge effects that cause shear and moment should be included in the analysis and design. Specified compressive strength of concrete \( f'_c \) should not be less than required in Section 3.2.2.2 nor greater than 5000 psi (34 MPa).

4.9.3.2 Thickness — The minimum thickness \( h \) of a uniform thickness dome should be computed by Eq. (4-24) using any consistent set of units. Buckling effects should be considered when the radius to thickness ratio exceeds 100

\[
h = 1.5 R f'_{c} \frac{w_u}{\phi f'_{c}} \text{ not less than 8 in. (200 mm) } \tag{4-24}
\]

where \( w_u \) and \( f'_c \) are expressed in the same units, and \( h \) and \( R \) are expressed in in. (mm).

The factored distributed \( w_u \) is the mean dead and water load (Load Combination U1.1). The strength reduction factor \( \phi \) is 0.7.

4.9.3.3 Minimum reinforcement — Reinforcement area on each face in orthogonal directions should not be less than 0.002 times the gross concrete area. Where tensile reinforcement is required by analysis the minimum reinforcement should conform to Section 4.3.3.

4.9.3.4 Crack control — Distribution of tension reinforcement required by analysis should conform to Section 4.4.2.

4.9.4 — Suspended steel floors

Steel floor tanks utilize a suspended membrane steel floor, generally with a steel skirt and grouted base plate to transfer tank loads to the concrete support structure, and a
steel compression ring to resist internal thrust forces. Design of suspended steel floors, and associated support skirts, base plates, and compression rings is part of the steel tank design (Section 4.1.3).

4.10—Concrete-to-tank interface

4.10.1 General

4.10.1.1 Scope—This section covers design of the interface region of concrete-pedestal elevated tanks.

4.10.1.2 Interface region—The interface region includes those portions of the support wall, tank floor, ringbeam, and steel tank affected by the transfer of forces from the tank floor and steel tank to the support wall.

4.10.1.3 Details—The details at the top of the support wall are generally proprietary and differ from one manufacturer to another. The loads and forces acting at the interface, and specific requirements are covered in Sections 4.10.3 through 4.10.5.

4.10.2 Design considerations

4.10.2.1 Load effects—The following load effects in combination with dead and live loads should be considered in design of the interface region:
(a) Loading caused by varying water level;
(b) Seismic and wind forces that cause unsymmetrical reactions at the interface region;
(c) Construction loads and attachments that cause concentrated loads or forces significantly different than the dead and water loads;
(d) Short and long-term translation and rotation of the concrete at the interface region, and the effect on the membrane action of the steel tank;
(e) Eccentricity of loads, where the point of application of load does not coincide with the centroid of the resisting elements;
(f) Effect of restrained shrinkage and temperature differentials;
(g) Transfer of steel tank loads to the concrete support structure;
(h) Anchorage attachments when required for uplift loads.

4.10.2.2 Analysis—Analysis should be by finite difference, finite element, or similar analysis programs that accurately model the interaction of the intersecting elements. The analysis should recognize:
(a) The three-dimensional nature of the problem;
(b) The non-linear response and change in stiffness associated with tension and concrete cracking, and the redistribution of forces that occur with stiffness changes;
(c) The effect of concrete creep and shrinkage on deformations at the interface;
(d) The sensitivity of the design to initial assumptions, imperfections, and construction tolerances. Appropriate allowance for variations arising from these effects should be included in the analysis.

4.10.3—Dome floors

4.10.3.1 Design considerations—The interface region should be analyzed for in-plane axial forces, radial and tangential shear, and moment for all loading conditions. Eccentricity arising from geometry and accidental imperfections in the construction process should be included in the analysis.

Various stages of filling, and wind and seismic overturning effects should be considered when determining the design loads. Particular attention should be given to the radial shear and moment in shell elements caused by edge restraint effects.

4.10.3.2 Ringbeam compression—The maximum service load compression stress in the ringbeam due to direct horizontal thrust forces should not exceed 0.30\(\sigma'_{fc}\).

4.10.3.3 Fill concrete—Concrete used to connect the steel tank to the concrete support structure should have a specified compressive strength not less than the concrete to which it connects or the design compressive strength, whichever is greater.

4.10.4—Slab floors

The support wall, tank floor, and steel tank should be analyzed for in-plane axial forces, radial shear, and moment for all loading conditions. The degree of fixity of the steel tank to the tank floor should be considered.

4.10.5—Suspended steel floors

4.10.5.1 Design considerations—The analysis and design of the concrete support element should include consideration of the following loading effects:
(a) Vertical loads not centered on the wall due to construction inaccuracies causing shear and moment at the top of the wall. Non-symmetrical distribution of eccentricities;
(b) Horizontal shear loads caused by an out of plumb skirt plate, or temperature differences between the steel tank and concrete wall;
(c) Transfer of wind and seismic forces between the tank and concrete support;
(d) Local instability at the top of the wall.

4.10.5.2 Support wall—The area near the top of the wall must have adequate shear strength and be adequately reinforced for the circumferential moments caused by the loads in Section 4.8.4.

4.10.5.3 Concrete support for base plates—The design centerline of the support wall and steel skirt should coincide. A concrete ringbeam having a nominal width and height at least 8 in. (200 mm) greater than the support wall thickness \(h\) is recommended for support of base plates. The concrete ringbeam may be omitted when the following conditions are met:
(a) The wall thickness \(h\) is equal to or greater than the width determined by

\[ h = b_p + 0.004d_w + b_e \]  

(4-25)

where all dimensions are expressed in in. (mm).

The edge distance term \(b_e\) should conform to Section 4.10.5.4, and the effective base plate width \(b_p\) to Section 4.10.5.5. The term 0.004\(d_w\) is the diameter tolerance of the wall in Section 3.6.1(a).

(b) Special construction control measures are implemented to ensure that the diameter and curvature of steel tank matches the concrete construction.

(c) The as-built condition is checked and documented. The radial deviation of the steel skirt and effective base plate centerlines from the support wall centerline should not be greater than 10 percent of the support wall thickness \(h\). The
as-built distance from edge of base plate to edge of concrete should not be less than 1.5 in. (40 mm).

4.10.5.4 **Base plate edge distance**—The combined inside and outside base plate edge distances $b_e$ in Eq. (4-25) should not be less than 6 in. (150 mm). If demonstrated construction practices are employed that result in an accurate fit of the steel tank to the concrete construction, the term $b_e$ in Eq. (4-25) may be reduced to not less than 3 in. (75 mm). Measurements and documentation of the as-built condition are required to demonstrate conformance to Section 4.10.5.3(c).

4.10.5.5 **Base plate**—The effective base plate width $b_p$ should be sized using a maximum design bearing strength of 2000 psi (14 MPa) for factored loads. The minimum effective base plate width $b_p$ is the larger of four times the nominal grout thickness or 4 in. (100 mm). The base plate width should not be less than the effective base plate width and should be symmetrical about the centerline of the steel skirt plate. A minimum base plate width of 6 in. (150 mm) symmetrical about the steel skirt plate centerline is recommended.

4.10.5.6 **Base plate grout**—Grout supporting the base plate should have a specified compressive strength not less than the supporting concrete or the design compressive strength, whichever is greater.

4.10.5.7 **Anchorage**—A positive means of attachment should be provided to anchor the steel tank to the concrete support structure. The anchorage should be designed for uplift forces and horizontal shear. The anchorage provided should not be less than 1 in. (25 mm) diameter anchor bolts at 10 ft (3 m) centers, or equivalent uplift capacity.

4.10.5.8 **Drainage**—A positive means of diverting rain and condensate water away from the grouted base plate should be provided. The drainage detail should incorporate a drip edge attached to the steel tank that diverts water away from the concrete support structure.

4.10.6 **Reinforcement details**—Reinforcement in concrete elements in the interface region should be sufficient to resist the calculated loads, but should not be less than the following.

(a) The minimum reinforcement ratio $\rho_g$ should not be less than 0.0025 in regions of compression and low tension stress;

(b) Where tension reinforcement is required by analysis the minimum reinforcement should conform to Section 4.3.3;

(c) Distribution of tension reinforcement required by analysis should conform to Section 4.4.2.

4.11—**Foundations**

4.11.1—**General**

4.11.1.1 **Scope**—This section covers structural requirements for foundations used for concrete-pedestal tanks. Geotechnical requirements are described in Section 4.12.

4.11.1.2 **Definitions**—Certain terms used in this section and Section 4.12 are defined as follows:

- **Shallow foundation**—Annular ring or raft foundation having a depth of embedment less than the foundation width. Load carrying capacity is by direct bearing on soil or rock; friction and adhesion on vertical sides are neglected.

- **Annular ring foundation**—A reinforced concrete annular ring whose cross-sectional centroid is located at or near the centerline radius of the concrete support wall and is supported directly on soil or rock.

- **Raft foundation**—A reinforced concrete slab supported directly on soil or rock, generally having a bearing area larger than an annular ring foundation.

- **Deep foundation**—Piles or piers and the pile or pier cap that transfer concrete support structure loads to a competent soil or rock stratum by end bearing, by mobilizing side friction or adhesion, or both.

- **Pile or pier**—Driven piles, drilled piles, drilled piers (caissons).

- **Pile or pier cap**—The concrete ring that transfers load from the concrete support structure to the supporting piles or piers.

4.11.1.3 **Foundation types**—Shallow and deep foundations used for support of concrete-pedestal elevated tanks are shown in Fig. 4.11.1.
4.11.3 Overturining—The foundation should be of sufficient size and strength to resist overturning forces resulting from wind, seismic, and differential settlement loads. Where high groundwater occurs, the effects of buoyancy should be included. The stability ratio (ratio of the resisting moment to overturning moment) should be greater than 1.5 for service load forces.

4.11.3.1 Resisting gravity load—The gravity service load \( P_s \) resisting overturning is the dead load \( D \) for wind loading, and dead plus water loads \( D + F \) for seismic loading.

4.11.3.2 Shallow foundations—The resisting moment is the product of the gravity service loads \( P_s \) and the distance from the foundation centerline to the centroid of the resisting contact pressure. The resisting contact pressure should not exceed the ultimate bearing capacity \( q_r \) defined in Section 4.12.4.

4.11.3.3 Deep foundations—The resisting moment is the product of the gravity service loads \( P_s \) and the distance from the foundation centerline to the centroid of the resisting group of piles or drilled piers. The maximum load acting on a deep foundation unit should not exceed the ultimate capacity \( Q_u \) defined in Section 4.12.5. Where piles or piers are capable of resisting tension loads and are connected to the superstructure, the tension capacity may be considered in assessing the stability ratio.

4.11.4—Shallow foundations

4.11.4.1 Annular ring foundation—Torsional effects and biaxial bending should be considered when the centroid of the footing and the centerline of the wall do not coincide. The footing may be designed as a one-way beam element that is a sector of an annulus when the centroids coincide and the circumferential biaxial effects may be excluded.

4.11.4.2 Raft foundation—The portion inside the concrete support wall is designed as a two-way slab and the cantilever portion may be designed as a one-way strip, or as a continuation of the two-way interior slab.

4.11.5—Deep foundations

4.11.5.1 Structural design—The structural design of piles or piers should be in accordance with national and local building codes. Recommendations for design and construction of drilled piers are found in ACI 336.3R.

4.11.5.2 Lateral load effects on piles or piers—The effect of lateral loads should be considered in the structural design of piles or piers.

4.11.5.3 Lateral load effects on pile or pier caps—The pile or pier cap should be designed for the shear, torsion, and bending moments that occur when piles or drilled piers are subject to lateral loads.

4.11.5.4 Lateral seismic loads—Piling should be designed to withstand the maximum imposed curvature resulting from seismic forces for freestanding piles, in loose granular soils and Soil Profile Types E and F. Piles subject to such deformation should conform to Section 9.4.5.3 of ASCE 7.

4.11.6—Design details

4.11.6.1 Load transfer—Forces and moments at the base of concrete wall should be transferred to the foundation by bearing on concrete, by reinforcement and dowels, or both. The connection between the pile or pier cap and piles or piers should be designed for bearing, shear, and uplift forces that occur at this location.

4.11.6.2 Development of reinforcement—Flexural steel should be checked for proper development at all sections. Hooks may be used to develop footing reinforcement where the footing extension is relatively small.

4.11.6.3 Serviceability—The service load tension reinforcement steel stress \( f_s \) at sections of maximum moment should not exceed 30,000 psi (205 MPa) for load case S1.1, dead and water loads only. Alternatively, sections of maximum moment should conform to the requirements of Section 4.4.2.

4.11.6.4 Sloped foundations—When tapered top surfaces are used, actual footing shape should be used to determine shear and moment capacities.

4.11.6.5 Concrete cover—The actual clear distance between the edge of foundation and edge of a pier or pile should not be less than 3 in. (75 mm) after installation.

4.11.6.6 Support wall openings—The local effects at large openings in the concrete support wall should be considered when the distance from the top of the foundation to the bottom of the opening is less than one-half the opening width. The foundation should be designed for the redistribution of loads across the unsupported opening width.

4.11.6.7 Seismic design details—Where design for seismic loads is required, details of concrete piles and concrete filled piles should conform to the requirements of Section A9.4.4.4 of ASCE 7.

4.12—Geotechnical recommendations

4.12.1—General

4.12.1.1 Scope—This section identifies the minimum requirements related to foundation capacity and settlement limits.

4.12.1.2 Geotechnical investigation—A subsurface investigation should be made to the depth and extent to which the tank foundation will significantly change the stress in the soil or rock, or to a depth and extent that provides information to design the foundation. The investigation should be by a qualified design professional.

The following information should be provided to the design professional responsible for conducting the geotechnical investigation:

(a) Tank configuration, including support wall diameter;
(b) Gravity loads acting on the foundation: dead, water, and live loads;
(c) Wind and seismic overturning moments and horizontal shear forces acting at top of foundation;
(d) Minimum foundation depth for frost penetration or to accommodate piping details;
(e) Whether deep foundation units are required to resist tension uplift forces.

4.12.1.3 Foundation requirements—The design of foundations should be based on the results of the geotechnical investigation. The foundation should be configured in accordance with the requirements of Sections 4.12.2 through 4.12.5. Structural components should conform to Section 4.11.
4.12.2 Foundation depth—Foundation depth should be below the extreme frost penetration depth, or as required by the applicable building code. A smaller foundation depth may be used if the foundation overlies material not susceptible to frost action. The minimum depth should be 12 in. (300 mm).

4.12.3 Settlement limits—The combined foundation and concrete support structure provide a very rigid construction that will experience little or no out-of-plane settlement. The subsurface deformations that require consideration are total settlement, and differential settlement causing tilting of the structure. Typical long-term predicted settlement limits under load combination S1.1, dead and full water load, are:

- Total settlement for shallow foundations: 1.5 in. (40 mm)
- Total settlement for deep foundations: \( \frac{3}{4} \) in. (20 mm)
- Tilting of the structure due to non-uniform settlement: 1/800.

Larger differential tilt is permitted when included in Eq. (4-1b). Maximum tilt should not exceed 1/300.

Elevations for slabs on grade, driveways, and sidewalks should be selected to have positive drainage away from the structure after long term settlements have occurred.

4.12.4—Shallow foundations

4.12.4.1 Ultimate bearing capacity—The ultimate bearing capacity \( q_u \) is the limiting pressure that may be applied to the soil/rock surface by the foundation without causing a shear failure in the material below the foundation. It should be determined by the application of generally accepted geotechnical and civil engineering principles in conjunction with a geotechnical investigation.

4.12.4.2 Allowable bearing capacity—The allowable bearing capacity \( q_a \) is the limiting service load pressure that may be applied to the soil/rock surface by the foundation. It should be the smaller value determined from:

(a) Permissible total and differential settlements;
(b) Ultimate bearing capacity divided by a safety factor not less than three.

4.12.4.3 Net bearing pressure—Ultimate or allowable bearing pressure should be reported as the net bearing pressure defined in Fig. 4.12.4.

4.12.4.4 Foundation size—The size of shallow foundations should be the larger size determined for settlement in accordance with Section 4.12.4.2 or the bearing capacity of the soil using the unfactored loads in Section 4.2.4.

4.12.5—Deep foundations

4.12.5.1 Ultimate capacity—The ultimate capacity of piles or piers \( Q_u \) should be based on a subsurface investigation by a qualified geotechnical design professional, and one of the following:

(a) Application of generally accepted geotechnical and civil engineering principles to determine the ultimate capacity of the tip in end bearing, and the side friction or adhesion;
(b) Static load testing in accordance with ASTM D 1143 of actual foundation units;
(c) Other in-situ load tests that measure end bearing and side resistance separately or both;
(d) Dynamic testing of driven piles with a pile driving analyzer.

4.12.5.2 Allowable capacity—The allowable service load capacity \( Q_a \) is the ultimate capacity \( Q_u \) divided by a safety factor not less than shown in Table 4.12.5. It should not be greater than the load causing the maximum permissible settlement.

4.12.5.3 Settlement and group effects—An estimate of the settlement of individual piles or piers and of the group should be made by the geotechnical design professional.

4.12.5.4 Lateral load capacity—The allowable lateral load capacity of piles and drilled piers and corresponding deformation at the top of the pile or pier should be determined by the geotechnical design professional. The subgrade modulus or other soil parameters suitable for structural design of the pile or pier element should be reported.

4.12.5.5 Number of piles or drilled piers—The number of piles or drilled piers should be the larger number determined for settlement in accordance with Section 4.12.5.2 or for the resistance of the soil or rock using the unfactored loads in Section 4.2.4.

**Table 4.12.5—Factor of safety for deep foundations**

<table>
<thead>
<tr>
<th>Ultimate capacity in accordance with Section</th>
<th>Recommended minimum safety factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.12.5.1(a)</td>
<td>3.0</td>
</tr>
<tr>
<td>4.12.5.1(b)</td>
<td>2.0</td>
</tr>
<tr>
<td>4.12.5.1(c)</td>
<td>2.0</td>
</tr>
<tr>
<td>4.12.5.1(d)</td>
<td>2.5</td>
</tr>
</tbody>
</table>
4.12.5.6 Spacing of piles or drilled piers—The minimum spacing between centers of driven piles should not be less than 2.0 times the butt diameter. The following should be considered when determining the spacing and arrangement of piles and drilled piers:
(a) The overlap of stress between pile or drilled pier units influencing total load capacity and settlement;
(b) Installation difficulties, particularly the effects on adjacent piles or drilled piers.
4.12.5.7 Number and arrangement of piles or drilled piers—The number and arrangement of piles or piers should be such that the allowable capacity \( Q_{a} \) is not exceeded when the foundation is subjected to the combined service loads defined in Section 4.2.4.
4.12.6—Seismic requirements
4.12.6.1 Site factor—In areas where \( A_{y} \geq 0.05 \) the geotechnical design professional should classify and report the soil type conforming Table 4.7.4.
4.12.6.2 Liquefaction potential—In areas where \( A_{y} \geq 0.05 \) the geotechnical design professional should report whether or not there is a potential for soil liquefaction at the site. Where soil liquefaction is possible, deep foundations, soil improvement, or other means should be employed to protect the structure during an earthquake.
4.12.7—Special considerations
4.12.7.1 Sloping ground—Where the foundation is on or near sloping ground the effect on bearing capacity and slope stability should be considered in determining bearing capacity and foundation movements.
4.12.7.2 Geologic conditions—Geologic conditions such as karst (sinkhole) topography, faults or geologic anomalies should be identified and provided for in the design.
4.12.7.3 Swelling and shrinkage of soils—Where swelling or shrinkage movements from changes in soil moisture content are encountered or known to exist, such movement should be considered.
4.12.7.4 Expanding or deteriorating rock—Where rock is known to expand or deteriorate when exposed to unfavorable environmental conditions or stress release, the condition should be provided for in the design.
4.12.7.5 Construction on fill—Acceptable soil types, and compaction and inspection requirements should be investigated and specified when foundations are placed on fill.
4.12.7.6 Groundwater level changes—The effect of temporary or permanent changes in groundwater levels on adjacent property should be investigated and provided for in the design.

CHAPTER 5—APPURTENANCES AND ACCESSORIES
5.1—General
5.1.1 Scope—This chapter describes the appurtenances required for operation and maintenance, and accessories commonly furnished with concrete-pedestal elevated tanks, as shown in Fig. 5.1. Items furnished at any given installation will depend on the project documents and the applicable building code.
5.1.2 Design—Design and detailing of accessories and appurtenances should conform to the applicable building code, and state and federal requirements where applicable. Loads should not be less than those required by ASCE 7. Dimensions and sizes where shown are intended to indicate what is commonly used, and may not conform to codes and regulations in all cases because of differences between codes and regulations and revision of these documents.
5.1.3 Personnel safety—The design and details of ladders, stairways, platforms, and other climbing devices should conform to OSHA and applicable building code requirements for industrial structures. The design and use of anti-fall devices (cages and safe clamp devices) should be compatible with the climbing system to which they attach. Attachment of ladders, stairways, platforms, and anti-fall devices to the structure should be designed to mechanically fasten securely to the structure during the anticipated service life, considering the exposure of the attachment to the environment.
5.1.4 Galvanic corrosion—Dissimilar metals should be electrically isolated to prevent galvanic corrosion.

5.2—Support wall access
5.2.1 Exterior doors—One or more exterior doors are required for access to the support wall interior, and should conform to the following:
(a) At least one personnel or vehicle door of sufficient size to permit moving the largest equipment or mechanical item through the support wall;
(b) Steel pipe bollards should be provided at the sides of vehicle door openings for impact protection;
(c) Doors at grade should have locking devices to prevent unauthorized access to ladders and equipment located inside the support wall.
5.2.2 Painters’ access—A hinged or removable door at the top of the support wall is required for access to the outside painters rigging from the upper platform. The opening should have a least dimension of 24 in. (610 mm). It may be screened and louvered to satisfy all or part of the vent area requirements.

5.3—Ventilation
5.3.1—Support wall vents
5.3.1.1 Location and number—The location and number of vents for ventilation of the concrete support wall interior should conform to state and local building code requirements based on occupancy classification. A removable vent at the top of the support wall may be used for access to the exterior rigging rails located at the tank/pedestal intersection.
5.3.1.2 Description—Vents should be stainless steel or aluminum, and should have removable insect screens.
5.3.1.3 Access—Vents should be accessible from the interior ladders, platforms or floors.
5.3.2—Tank vent
5.3.2.1 Location—The tank vent should be centrally located on the tank roof above the maximum weir crest elevation.
5.3.2.2 Description—The vent consists of a support frame, screened area and cap. The support should be fastened to a flanged opening in the tank roof. The vent cap should be provided with sufficient overhang to prevent the entrance of wind driven debris and precipitation. A minimum of 4 in. (100 mm) should be provided between the roof surface and the vent cap. The vent should be provided with a bird screen. Insect
screening should be provided when required by applicable building codes, health regulations, or project documents.

5.3.2.3 Capacity—The tank vent should have an intake and relief capacity sufficiently large that excessive pressure or vacuum will not be developed when filling or emptying the tank at maximum flow rate of water. The maximum flow rate of water exiting the tank should be based on an assumed break in the inlet/outlet at grade when the tank is full. The overflow pipe should not be considered as a vent. Vent capacity should be based on open area of screening used. Vents should be designed to operate when frosted over or otherwise clogged, or adequate pressure/vacuum relief should be provided.

5.3.3 Pressure/vacuum relief—A pressure/vacuum relief mechanism should be provided that will operate in the event of tank vent failure. It should be located on the tank roof above the maximum weir crest elevation and may be part of the vent. Design of the pressure/vacuum relief mechanism should be such that it is not damaged during operation, and that it returns to the normal position after relieving the pressure differential.

5.4—Steel tank access

5.4.1 General—Access from interior of the support wall to the tank roof and interior is provided by the appurtenances described below.

5.4.1.1 Materials—Materials should conform to the following:
(a) Ladders and platforms: painted or galvanized steel;
(b) Access tube: painted steel plate or pipe;
(c) Roof hatches and covers: painted or galvanized steel, or aluminum;
(d) Manholes: painted or galvanized steel, or stainless steel;
(e) Embedments: painted or galvanized steel, or stainless steel.

5.4.1.2 Attachment to steel tank—Attachment of ladders and other accessories to the steel tank should be with brackets welded to the steel tank. Attachment of the accessory to the brackets may be by welding or bolting. The access tube exterior should be reinforced where ladder brackets are attached so that potential ice damage is confined to the ladder and bracket and not the access tube shell.

5.4.1.3 Attachment to concrete—The following methods are commonly used for attachment of accessories and appurtenances to concrete:
(a) Embedded anchor bolts and threaded anchorages;
(b) Welding or bolting to embedment plates anchored with headed studs;
(c) Drilled anchors: expansion type and grouted type using chemical adhesives. Only drilled anchors that can be inspected or tested for proper installation should be used.

5.4.2—Ladders
5.4.2.1 Location—Interior vertical access ladders should be provided at the following locations:
(a) Grade to upper platform;
(b) Upper platform to tank floor manhole;
(c) Upper platform to steel tank roof (mounted on access tube interior);
(d) Steel tank roof to steel tank interior floor (mounted on access tube exterior). In cold climates this ladder may be omitted to prevent damage from ice.

5.4.2.2 Fall protection—A safe climbing device should be provided wherever cages or other means of fall protection are not provided. A safe climbing device is recommended for access tube ladders. Ladders with safe climbing devices should be continuous through intermediate or rest platforms so that personnel are not forced to disengage. Extension rails may be required at manhole openings.

5.4.3—Platforms and handrails
5.4.3.1 Loads—Platforms and handrails should be designed for the minimum loads defined in Sections 4.2 through 4.4 of ASCE 7, and the requirements of OSHA and the applicable building code.

5.4.3.2 Platforms—Platforms should be provided at the following locations, complete with handrails and toeboards.
(a) An upper platform located below the tank floor that provides access from the wall ladder to: the access tube interior, the tank floor manhole, and the painters support wall. The access tube provides access from the wall ladder to: the access tube interior;
(b) Intermediate platforms are used for access to piping or equipment, as rest platforms, and with offset ladders. A least platform dimension of 3 ft (0.9 m) is recommended.

5.4.3.3 Roof handrail—A handrail surrounding the roof manholes, vents, and other roof equipment should be provided. Where a handrail is not used or where equipment is located outside the handrail, anchorage devices for the attachment of safety lines should be provided.

5.4.4 Access tube—The access tube provides interior access to the steel tank roof from the upper platform, through the water containing portion of the tank. The access tube should be not less than 42 in. (1.07 m) diameter, equipped with a hinged hatch cover with inside handle and interior locking device. The hatch opening size should have a least dimension of 24 in. (610 mm) or larger.

5.4.5 Steel tank roof openings—Roof openings should be located above the overflow level at ladder locations. Safety grating, barricades, warning signs, or other protection should be provided at roof openings to prevent entry at locations where ladders are omitted. Hatches should be weatherproof and equipped with a hasp to permit locking. Roof hatch openings should have a least dimension of 24 in. (610 mm) or larger.

5.4.6 Tank floor manhole—A manhole in the tank floor should be provided which is accessible from the upper platform or from a ladder that extends from the platform to the opening. It should have a least dimension of 24 in. to 30 in. (610 to 760 mm).

5.5—Rigging devices
Bar, tee rails, or other rigging anchorage devices should be provided for painting and maintenance of the structure. The safe load capacity for rigging devices should be shown on construction drawings. Access to rigging attachments should be provided.

5.5.1 Exterior rails—A continuous bar or tee rail near the top of the exterior of the concrete support structure should be provided. The rail may be attached to the support wall or steel tank. Access to the rail is from the upper platform through a painters opening ( Section 5.2.2 ).

5.5.2 Tank interior—Provision for painting the interior of steel tanks should be provided. Painters rails attached to the roof or pipe couplings with plugs in the roof are commonly used for rigging attachment.

5.5.3 Support wall interior—Rigging attachments should be provided near the top of the support wall for inspection and maintenance of piping and equipment not accessible from platforms or floors.

5.6—Above ground piping
5.6.1 Materials—Steel and stainless steel pipe and fittings may be used for above ground piping.

5.6.1.1 Minimum thickness—Only steel pipe with a minimum thickness of 1/4 in. (6.4 mm) should be used where pipe is exposed to stored water inside the tank. Minimum thickness of pipe located outside the stored water area should be:
(a) Steel pipe without interior lining or coating: 1/4 in. (6.4 mm);
(b) Steel pipe with interior lining or coating: 3/16 in. (4.8 mm);
(c) Stainless steel pipe: 12 gage (2.7 mm);

5.6.1.2 Interior linings or coatings—Where interior linings or coatings are required, pipe components should be detailed and field assembled so as not to damage the interior lining or coating.

5.6.2—Inlet/outlet pipe
5.6.2.1 Configuration—Usually a single inlet/outlet pipe is used to connect the tank to the system water main. The
the entrance capacity of the overflow pipe is not adequate.

5.6.2.2 Sizing—The minimum diameter of the inlet/outlet pipe is based on acceptable losses due to system flows and consideration of freezing potential.

5.6.2.3 Support—Vertical pipe loads, including axial expansion joint forces, are supported at the tank floor. The weight of water in the pipe is supported by the base elbow or piping below the expansion joint. Pipe guides for horizontal support are attached to the support wall at intervals that should not exceed 20 ft (6 m).

5.6.2.4 Expansion joint—The expansion joint in the inlet/outlet pipe should be designed and constructed to accommodate any differential movement caused by settlement and thermal expansion and contraction. The required flexibility should be provided by an expansion joint located near grade in the vertical section of pipe.

5.6.2.5 Differential movement—Potential movement between the water main system and tank piping due to settlement or seismic loads should be considered in the design. A mechanical joint or coupling should be provided at the point of connection to the water main system unless no movement is expected. Additional couplings or special fittings may be used if differential movement is expected to be large.

5.6.2.6 Entrance details—Flush mounted inlet/outlet pipe should have a removable silt stop at or below the design low water level that projects a minimum of 6 in. (150 mm) above the tank floor liner. Inlet safety protection should be provided in accordance with applicable safety regulations. Where no permanent protection is required a safety grate or plate should be provided during construction.

5.6.3 — Overflow

5.6.3.1 Configuration—The top of the overflow should be located within the tank at the level required by the project documents and should run approximately as shown in Fig. 5.1. The discharge should be designed such that it will not be obstructed by snow or other objects. The horizontal run of pipe below the tank floor should be sloped for positive drainage. In cold climates the overflow may be located on the interior of the access tube if there is potential for ice damage.

5.6.3.2 Sizing—The overflow pipe should be sized to carry the maximum design flow rate of the inlet pipe. Head losses from pipe, fittings, and exit velocity should be considered in determining pipe diameter. The overflow pipe should not be less than 4 in. (100 mm) diameter.

5.6.3.3 Entrance—The entrance to the overflow pipe should be designed for the maximum inlet pipe flow rate, and should have a vortex prevention device. The design should be based on the water level cresting within 6 in. (150 mm) above the overflow level. A suitable weir should be provided when the entrance capacity of the overflow pipe is not adequate.

5.6.3.4 Support—Supports for the overflow pipe should be designed for static, dynamic, and thermal loads. Support brackets, guides and hangers should be provided at intervals not exceeding 20 ft (6 m). The overflow and weir section within the steel tank may be attached to the access tube for support.

5.6.3.5 Discharge—The overflow pipe should discharge onto a splash block at grade, or into a sump or a drain line, that effectively removes water away from the foundation. The end of the overflow pipe should be covered with a coarse, corrosion resistant mesh or a flap valve.

5.6.4 Tank drain—An inlet/outlet pipe or a separate drain line that is flush with the low point of the tank should be provided to completely drain the tank.

5.7 — Below ground piping and utilities

5.7.1 Pipe cover—Pipe cover should be greater than the extreme frost penetration, or as required by the applicable building code. The minimum cover should be 24 in. (600 mm).

5.7.2 Differential movement—Connecting piping and utilities should have sufficient flexibility to accommodate twice the predicted settlement or movement due to seismic loads without damage.

5.8 Interior floors

5.8.1 General—A concrete slab on grade should be provided inside the concrete wall. One or more intermediate floors above grade may be furnished when provided for in the original design.

5.8.1.1 Occupancy classification—Each portion of the interior space should be classified according to its use or the character of its occupancy and the requirements of the applicable building code for the type of occupancy should be met.

5.8.1.2 Posted live loads—The safe floor live loads should be displayed on a permanent placard in a conspicuous location at each floor level.

5.8.2 Slabs-on-grade—Refer to ACI 302.1R for guidance on floor slab construction and ACI 360 for recommended design requirements.

5.8.2.1—Slabs on grade are usually designed as plain concrete slabs where reinforcement, as well as joint spacing, are used to control cracking and to prevent cracks from opening. Where project documents do not indicate how the slab on grade will be used, the values in Table 5.8.2 are recommended.

Table 5.8.2—Minimum requirements for slabs on grade

<table>
<thead>
<tr>
<th>Description</th>
<th>Door opening width less than 8 ft (2.4 m)</th>
<th>Door opening width greater than 8 ft (2.4 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete strength: $f_{c'}$</td>
<td>3500 psi (24 MPa)</td>
<td>4000 psi (28 MPa)</td>
</tr>
<tr>
<td>Thickness</td>
<td>5 in. (125 mm)</td>
<td>6 in. (150 mm)</td>
</tr>
<tr>
<td>Reinforcement ratio</td>
<td>0.0018</td>
<td>0.0018</td>
</tr>
</tbody>
</table>

Note: Floors intended to be used for parking of heavy vehicles or similar loads should be designed for the specific loading anticipated.

5.8.2.2 Details of reinforcement—Reinforcement should be located approximately 2 in. (50 mm) below the top surface of the slab. Slabs greater than 8 in. (200 mm) thick should have two layers of reinforcement. Either welded wire fabric or deformed bar reinforcement may be used. Maximum spacing of wires or bars should not be greater than 18 in. (460 mm). Reinforcement should be maintained in correct position by
support chairs or concrete blocks. Additional reinforcement should be provided at floor edges and other discontinuities, as required by the design.

5.8.2.3 Joints—The following joint types are commonly used and should conform to ACI 504R:

(a) Isolation joints—The floor slab should be separated structurally from other elements of the structure to accommodate differential horizontal and vertical movements. Isolation joints should be provided at junctions with walls, columns, equipment or piping foundations, and other points of restraint. Isolation joints should be formed by setting expansion joint material prior to concrete placement. The joint filler should extend the full depth of the joint and not protrude above the surface.

(b) Contraction joints—Joint spacing should be at 20 ft (6 m) maximum centers. Joints should be hand-tooled or saw cut to a depth of one-fourth to one-third times the slab thickness. Reinforcement should be continuous across the joint for slabs up to 70 ft (21 m) wide. For larger slabs the minimum reinforcement ratio should be increased by the ratio of the slab width to 70 ft (21 m). Alternatively dowels with discontinuous reinforcement can be provided at a spacing not exceeding 70 ft (21 m).

5.8.2.4 Drainage—The surface of slabs-on-grade should have a minimum slope of 1 percent sloping to drains. Slope to doorways where drains are not provided.

5.8.2.5 Subgrade—The suitability of in-situ and fill soils for supporting the slab on grade should be determined by the geotechnical design professional. Unsuitable soils should be improved or replaced. Any fill materials should be compacted to a density of 90 to 95 percent modified Proctor density (ASTM D 1557). Where expansive soils are encountered, the recommendations of the geotechnical design professional should be followed.

5.8.2.6 Structural floors—An isolated structural floor slab near grade may be required where compressible or expansive soils are encountered. Design of structural floors should conform to ACI 318.

5.8.3 Intermediate floors—One or more floors above grade may be constructed for storage or other uses. Typically the structural system is a flat slab, or a beam and slab system attached to the support wall, and may include intermediate columns.

5.8.3.1 Loads—Loads should conform to the applicable building code, based on occupancy classification. Floors used for storage should be designed for a minimum uniform live load of 125 lb/ft² (6 kPa). The minimum design live load should be 50 lb/ft² (2.4 kPa).

5.8.3.2 Design and construction—Dead and live loads from any intermediate floors should be accounted for in the design of the support wall and foundation. Localized axial loads, moments and shear due to beam end reactions should be considered in the design of the support wall.

5.9—Electrical and lighting

5.9.1 General—Electrical work should conform to the governing applicable building code and other applicable regulations.

5.9.2 Lighting and receptacles

5.9.2.1 Exterior—A single light should be provided above each personnel and vehicle door. These lights should be controlled by a single switch located on the interior of the support wall, adjacent to the open side of the personnel door.

5.9.2.2 Interior—Interior lighting and receptacles should be provided at the following locations:

(a) Base—Lights should be provided 8 ft (2.4 m) above the slab-on-grade at equal intervals not exceeding 30 ft (9 m) along the support wall. These lights should be controlled by a single switch located adjacent to the open side of the access door. One convenience outlet should be provided adjacent to the power distribution panel.

(b) Ladder/landing—Lights should be provided adjacent to the support wall access ladder at intervals not exceeding 25 ft (8 m). The lower light should be at 8 ft (2.4 m) above the slab and the top ladder light should be placed above the upper platform. A light should be provided 8 ft (2.4 m) above each intermediate platform. Lights should be provided at the top and bottom of the interior access tube. These lights should be controlled by a single switch located at the base of the support wall access ladder.

5.9.3 Obstruction lighting—Obstruction lighting and marking requirements depend on structure height and proximity to air traffic. The Federal Aviation Agency (FAA) should be contacted to determine if obstruction lighting is required. Obstruction lighting should be of weathertight, corrosion resistant construction, conforming to FAA standards.

CHAPTER 6—REFERENCES

6.1—Recommended references

The documents of various standards-producing organizations referred to in this document are listed below with their serial designation.

American Concrete Institute

ACI 116R Cement and Concrete Terminology
ACI 117 Standard Specifications for Tolerances for Concrete Construction and Materials
ACI 209R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
ACI 211.1 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
ACI 302.1R Guide for Concrete Floor and Slab Construction
ACI 304R Guide for Measuring, Mixing, Transporting, and Placing Concrete
ACI 305R Hot Weather Concreting
ACI 306R Cold Weather Concreting
ACI 308 Standard Practice for Curing Concrete
ACI 309R Guide for Consolidation of Concrete
ACI 315 Details and Detailing of Concrete Reinforcement
ACI 316 Building Code Requirements for Structural Concrete
ACI 336.3R Design and Construction of Drilled Piers
ACI 347R Guide to Formwork for Concrete
ACI 360 Design of Slabs on Grade
ACI 504R  Guide to Sealing Joints in Concrete Structures

American Society of Civil Engineers
ASCE 7  Minimum Design loads for Buildings and Other Structures

American Society for Testing and Materials
A 185  Standard Specification for Steel Welded Wire Fabric, Plain, for Concrete Reinforcement
A 497  Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement
A 615/A 615M Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A 617/A 617M Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
A 706/A 706M Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
C 33  Standard Specification for Concrete Aggregates
C 94  Standard Specification for Ready-Mixed Concrete
C 150  Standard Specification for Portland Cement
C 309  Specification for Liquid Membrane—Forming Compounds for Curing Concrete
C 595  Standard Specification for Blended Hydraulic Cements
C 803  Standard Test Method for Penetration Resistance of Hardened Concrete
C 900  Standard Test Method for Pullout Strength of Hardened Concrete
C 1074  Standard Practice for Estimating Concrete Strength by the Maturity Method
D 698  Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lb/ft, (600 kN-m/m)]
D 1143  Standard Test Method for Piles Under Static Axial Compressive Load
D 1557  Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort [56,000 ft-lb/ft, (2700 kN-m/m)]

American Welding Society
AWS D1.4  Structural Welding Code—Reinforcing Steel

The above publications may be obtained from the following organizations:

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

The American Society of Civil Engineers
345 East 47th Street
New York, N.Y. 10017-2398

American Society for Testing and Materials
100 Barr Harbor Drive
West Conshohocken, Penn. 19428.

American Welding Society
550 N.W. Le June Road
Miami, Fla. 33126

6.2—Cited references

APPENDIX A—COMMENTARY ON GUIDE FOR THE ANALYSIS, DESIGN, AND CONSTRUCTION OF CONCRETE-PEDESTAL WATER TOWERS

CHAPTER 1—GENERAL COMMENTARY

A1.1—Introduction
Since the 1970s concrete-pedestal elevated water-storage tanks have been constructed in North America with a steel water-containing element and an all-concrete support structure. The generic term “composite elevated tank” is often used to describe tanks of this configuration.

Concrete-pedestal tanks are competitively marketed as complete entities including design, and are constructed under design-build contracts using proprietary designs, details, and methods of construction. The designs are, however, frequently reviewed by owners and their consulting engineers, or by city or county officials.

Concrete-pedestal tanks designed and constructed in accordance with the recommendations of this guide can be expected to be durable structures that require only routine maintenance. A steel tank is used for containing the stored water. Details of concrete surfaces that promote good drainage and avoid low areas conducive to ponding, essentially eliminate the problems associated with cyclic freezing and thawing of wet concrete in cold climates. The quality of concrete for concrete-pedestal tanks in this document meets the requirements for durable concrete as defined in ACI 201.2R. It has adequate strength, a low water-cementitious material ratio, and air-entrainment for frost exposure. The concrete support structure loads are primarily compression with little or no cyclic loading with stress reversal.

A1.2—Scope
This document considers only concrete-pedestal elevated water-storage tanks of the types shown in Fig. 1.2. This makes it possible to address specific design and construction issues unique to these tank configurations. Concrete floor tanks generally have a tank diameter to support wall diameter ratio of 1.5 to 2.0, and for steel floor tanks it is usually less than 1.5.

A1.4—Terminology
Terminology in this document is consistent with the definitions in ACI 116R. The concrete cylindrical support wall
is commonly referred to as a concrete-pedestal or shaft. Neither term is an accurate description of the concrete cylindrical support wall, and term support wall or wall is used in this document.

CHAPTER 2—MATERIAL COMMENTARY
A2.1—General
The provisions of this section are intended to address requirements specific to the concrete-pedestal elevated tank that may limit or supplement ACI 318.

A2.2—Cements
Placement of concrete for a concrete support structure usually extends over a period of weeks. For concrete exposed to view, using the same brand and type of cement minimizes variations in concrete color of elements.

A2.4—Water
Water containing iron oxide that may cause staining should not be used with light colored concrete.

A2.6—Reinforcement
Welding of reinforcement is infrequent for concrete-pedestal tanks. Limiting welding to ASTM A 706/A 706M bars avoids the testing and inspection requirements.

CHAPTER 3—CONSTRUCTION COMMENTARY
A3.1—General
A3.1.1—The structural concrete construction provisions of this guide emphasize the construction requirements unique to concrete-pedestal tanks, and are to conform to ACI 318 except as modified. ACI 301 is recommended for use in preparing project specifications.

A3.1.2—Quality assurance requirements are defined, and are considered good practice. The requirements conform to ASCE 7 Section A.9.1.6, and are mandatory where seismic design is required. The design professional responsible for design is also responsible for the quality assurance plan necessary to verify that design requirements have been met. The contractor is responsible for establishing procedures for controlling the work.

A3.2—Concrete
A3.2.2 Concrete quality—The quality of concrete is intended to provide durable concrete in all climates.

A3.2.3.3—Admixtures are important components of a concrete mix that may provide beneficial modifications to concrete properties. Admixtures may affect more than one property of concrete, sometimes adversely affecting desirable properties. Manufacturer’s instructions and limitations should be followed and it is recommended that the effects of admixtures be evaluated through testing with representative materials and placement conditions prior to use in the structure. ACI 212.3R and 212.4R provide guidance for using chemical admixtures and high-range water-reducing admixtures.

A3.2.4—Concrete production

A3.2.4.1—High-range water-reducing admixtures can be added at the batch plant or the site. Short transit times and the ability to produce concrete with a consistent slump lend themselves to batch plant dispensing of high-range water-reducing admixtures. Long transit times and site personnel experienced in using high-range water-reducing admixtures lend themselves to site dispensing of the admixture.

A3.2.5 Placement—Contingency plans should be prepared to handle breakdown of equipment during concrete placement. At least one spare vibrator should be kept on the site during concrete placement operations. Concrete should be placed in such a manner as to avoid cold joints in the structural element being placed. Retarders should be used when required to prevent cold joints.

A3.2.6 Curing—Curing compounds are almost always used for this type of structure. Where subsequent coatings are to be applied to the concrete, the curing compound should be compatible with the coatings or the material should be removed prior to coatings application.

A3.2.7—Weather

A3.2.7.2—Insulated formwork is commonly used for protection of the support wall concrete during cold weather.

A3.2.8 Testing, evaluation and acceptance—When concrete fails to meet the acceptance criteria of ACI 318, structural analysis or additional testing, or both, should be performed to determine if the component is structurally adequate.

A3.2.8.1—Concrete placements can occur as often as once a day, and at 28-days most concrete is under subsequent placements that would have to be demolished if the lower concrete were found to be deficient and had to be replaced. To gain some assurance that the 28-day tests will meet the strength requirements extra cylinders are usually made to provide an early-age strength that can be used to estimate 28-day strength.

A3.2.9—Joints and embedments

A3.2.9.1—Construction joints are typically limited to horizontal joints in the wall and circumferential joints in the dome. Bonding agents and formed keyways are not normally required for horizontal construction joints in the wall. Vertical construction joints are not normally used because of the relatively small form area required to form a lift.

A3.3—Formwork

A3.3.1 General—ACI 347R provides detailed information on formwork design, construction and materials. The design of the formwork system for the concrete support structure must incorporate safety features required by state and federal safety standards.
A3.3.1.1—The support wall forms are typically re-used a large number of times, and a durable facing material is required in order to maintain uniformity of the surface.

A3.3.2—Foundations

A3.3.2.3—Side forms are typically removed the day following concrete placement. Where adequate protection from cold weather is provided, an elapsed time of 12 hr will generally provide concrete strength significantly in excess of that required for safe form removal.

A3.3.3—Support wall

A3.3.3.1—The support wall is subjected to large compressive forces and generally requires a high degree of accuracy with regard to shell tolerance. Properly designed jump forms with through ties can routinely achieve the required tolerances. Vertical alignment should be controlled with laser equipment. Wall forms should be designed for the full concrete head to avoid overloading and excessive deflection that can occur when forms designed for less than the full head are accidentally overfilled.

A3.3.3.2—The form deflection limits are those specified in ACI 303R for concrete exposed to view.

A3.3.3.3—A uniform pattern of horizontal and vertical rustications visually breaks up the surface of the support wall, and makes variations in surface color and texture less noticeable. Horizontal rustications provide shadow lines that make construction joint offsets less noticeable.

A3.3.3.5—Forms are typically removed the following day, provided the concrete has sufficient strength to permit form removal without damage to the concrete. A minimum concrete compressive strength of 800 psi (5.5 MPa) is generally adequate to prevent damage to concrete surfaces during wall form removal. Where supports or embedments are attached to the concrete for moving forms or other construction activities, the supports or embedments should not be used until the concrete has gained sufficient strength for their safe use.

A3.3.4—Tank floor

A3.3.4.2—Dome forms generally may be removed after 24 hr and flat slab forms at 72 hr when temperatures are above 50 F (10 C). Verification of concrete strength by one of the methods in Section 3.2.8.2 is recommended for form removal.

A3.4—Reinforcement

ACI 315 provides guidance for detailing reinforcement.

A3.5—Concrete finishes

A3.5.2 Formed surfaces—A smooth as-cast finish combined with a uniform rustication pattern results in a pleasing concrete surface, and is used for the majority of installations. Special form finishes should be limited to light sandblasting to enhance color uniformity. Rubbed and floated finishes are labor-intensive and are normally not used.

A3.5.3—Troweled finishes are defined in ACI 301.

A3.6—Tolerances

Tolerances for the support structure are based on the combined requirements of strength, construction technique, economic feasibility, and aesthetics.

A3.7—Foundations

A3.7.2—Earthwork

A3.7.2.2—Excavations for shallow foundation excavations should be inspected to ensure that the proper bearing stratum has been reached, and that conditions are consistent with the findings of the geotechnical investigation. The inspection should be by a qualified design professional familiar with the geotechnical report and the design requirements.

A3.7.2.4—Minimum compaction of 95 percent standard Proctor density is recommended where a sidewalk is placed on compacted backfill outside the support wall. Otherwise, backfill should be compacted to a minimum of 90 percent standard Proctor density to avoid potential settlement.

A3.7.3—Field inspection requirements are defined, and are considered good practice. The requirements conform to ASCE 7, and are mandatory where seismic design is required.

CHAPTER 4—DESIGN COMMENTARY

A4.1.2 Design of concrete support structure—The provisions of Chapter 19 of ACI 318 are the basis for analysis and design of shell elements of the concrete support structure. Applicable sections of Chapters 10, 11, 12, and 21 of ACI 318 are also incorporated into the design. Methods of analysis can include classical theory, simplified mathematical models, or numerical solutions using finite element, finite differences, or numerical integration techniques. The general analysis should consider the effects of restraint at the boundaries of shell elements.

A4.1.3 Design of tank—American Water Works Association standard AWWA D100 has been used for design of the steel portion of concrete-pedestal tanks. AWWA Committee D170 is currently writing a standard for design and construction of concrete-pedestal tanks that covers the entire structure.

A4.2—Loads

A4.2.1—ASCE 7 minimum design loads are adapted to concrete-pedestal elevated water-storage tanks. The loads are for Structure Classification Category IV defined in Table 1-1 of ASCE 7. The Category IV classification includes structures designated as essential facilities required in emergencies. The structure is considered an essential facility where the stored water is required for fire protection or in an emergency.

A4.2.2.3—The minimum roof live load of 15 lb/ft² (720 kPa) is commonly used with elevated tanks, and is slightly greater than the 12 lb/ft² (570 kPa) required by ASCE 7 to account for the presence of workmen and materials during repair and maintenance operations.

A4.2.2.5—Eccentricity of dead and water loads causes additional overturning moments that should be accounted for in the design. Eccentricity occurs when: (a) the tank is not concentric with the support wall, (b) the support wall is out-of-plumb, or (c) the foundation tilts because of differential settlement. The total eccentricity included in Eq. (4-1a) consists of a 1 in. (25 mm) allowance for tank eccentricity with respect to the support wall, plus an eccentricity of 0.25 percent times the height from bottom of foundation to top of support structure measured at the wall. The latter is intended to account for out-of-plumb construction and foundation tilt.
The combination of these effects is random, and the deviations implied by Eq. (4-1a) should not be used as construction tolerances.

It is assumed that half the minimum eccentricity in Eq. (4-1a) is due to tilting of the foundation (foundation tilt of 1/800). When a geotechnical investigation indicates that differential settlement across the foundation width is expected to be higher than that, then the additional tilt is to be included in determination of the vertical load eccentricity in Eq. (4-1b).

A4.2.2.7—Generally, concrete creep decreases the forces associated with restrained deformations at the boundaries of shell elements and at discontinuities.

Shrinkage generally causes cracking of the concrete support wall at restrained boundaries, such as the top of foundation, intermediate floor slabs, corners of openings, or locations where there are significant differences in concrete age of adjacent elements. Reinforcement is needed at these locations to control this cracking.

The detrimental effect of through-thickness and in-plane temperature differences is tension in the concrete that may cause cracking. Where minimum reinforcement is provided and where temperature differences are not excessive, thermal effects may be disregarded.

A4.2.3 Factor combinations—Load Combinations are divided into two groups based on whether they add to the effect of dead and water loads (Group 1), or whether they counteract the dead and water loads (Group 2).

Water has the characteristics of a dead load as well as a live load. It is like a dead load in that its magnitude is well defined, and like a live load in that the load is not necessarily permanent and may be applied repeatedly during the life of the structure. To account for the latter effect, the load factor for water is 1.6. This is suitable for elements in compression, but may result in excessive service-load stresses for flexural and tension elements. Section 4.4.2 is a crack control serviceability check that limits crack widths at service loads for flexure and direct tension. For flexural elements, such as concrete elements, the amount of reinforcement required may be controlled by these serviceability requirements.

The load combinations conform to ACI 318, except for combinations U1.4 and U2.2 which include seismic loads. These are ASCE 7 load combinations increased by a partial seismic load factor of 1.1, as required in ASCE 7 when ACI 318 factors are used. Load Combination U1.4 is the ASCE 7 basic load combination 1.2 D’ + 1.0E’ + 0.5L + 0.2S multiplied by \( \gamma_E \). The term D’ includes D + F, eccentricity G is included with L, and the snow load term 0.2S is not included because of its relatively small contribution to the total load. The term E’ represents the combined effects of horizontal seismic load E and vertical seismic load E’v. Vertical seismic load effects have generally not been included in design of elevated tanks and, therefore, the ability to continue to exclude them is accomplished by treating vertical seismic load effects as a separate load component E’v. Load Combination U2.2 is developed in a similar manner starting with ASCE 7 basic load combination 0.9D’ – 1.0E’ multiplied by \( \gamma_E \). Substituting D + F for D’ and E and E’v for E’ gives 0.99(D + F) – 1.1E – E’v. The structural effects T of differential settlement, creep, shrinkage or temperature effects are usually not significant, and have not been shown for clarity.

A4.2.3.4—The term E’v represents the vertical seismic load effect occurring in combination with the horizontal seismic load effect E. Historically, vertical seismic load effects have not been included in combined loads when designing elevated tanks, and where they have been included it has generally been by a square-root-of-sum-of-squares method rather than by direct summation. The entire subject of earthquake loads acting on non-building structures is undergoing extensive review at this time, and for this reason the committee decided to make no recommendation in regard to inclusion of vertical seismic loads in design. The design professional preparing the project documents should include requirements for vertical seismic loads where appropriate. In some instances this may require load combinations and load factors other than those in Eq. U1.4 and U2.2. Where vertical seismic loads are required to be in accordance with ASCE 7, the vertical load effect term is determined from Eq. (9.2.2.6-1) of ASCE 7, which results in \( E'v \) equal to \( \gamma_E \) (±0.5CvD) for concrete design.

A4.2.3.5—The partial seismic load factor \( \gamma_E \) of 1.1 for concrete elements is required by Section A9.6.1.1.1 of ASCE 7 to account for an incompatibility between the \( \phi \) factors of ACI 318 and the load factors in the ASCE 7 load combinations used in this document.

A4.2.4 Unfactored load combinations—Unfactored service load combinations are presented in a form comparable to factored loads. A reduction factor of 0.75 is used with wind and seismic loads in combination with gravity loads. The 0.75 is the reciprocal of 1.33, the allowable stress increase permitted with wind or seismic load combinations.

Load Combination S1.1 is the basic long-term load combination used to check serviceability requirements such as concrete cracking and foundation settlement.

The structural effects T of differential settlement, creep, shrinkage or temperature effects are usually not significant, and have not been shown for clarity.

A4.2.4.4—See A4.2.3.4 for discussion on including vertical seismic loads. When seismic loads other than those in ASCE 7 are to be used, it may be necessary to use load combinations other than those in Eq. S1.1 and S2.2 for allowable stress design.

A4.3 Strength requirements

A4.3.2 Design methods—The Strength Design Method of ACI 318 is the preferred method for design of concrete elements.

A4.3.3 Minimum reinforcement

A4.3.3.1—The minimum flexural reinforcement ratio of 3 \( \sqrt{f_y} / f_y \) in in.-lb units (0.25 \( \sqrt{f_y} / f_y \) in SI units) in the tension face is the same as required by ACI 318. This requirement is intended to prevent abrupt stress changes at the onset of cracking.

A4.3.3.2—The minimum reinforcement ratio of 5 \( \sqrt{f_y} / f_y \) in in.-lb units (0.42 \( \sqrt{f_y} / f_y \) in SI units) for regions of significant tension stress is based on equating the cracking strength of plain concrete to \( f_y \). The direct tension cracking strength is taken equal to two-thirds the modulus of rupture 7.5 \( \sqrt{f_y} \) in
in.-lb units \([5/8 \sqrt{f_c'}]\) in SI units. This requirement is intended to prevent abrupt strength changes when cracking occurs.

**A4.4—Serviceability requirements**

**A4.4.2 Control of cracking**—In slabs, locations of maximum tension steel stress occur at points of maximum moment, and at points where reinforcement is terminated. Flexural tension caused by restraint of deformations and direct tension can occur in the wall and the dome near the tank interface.

Eq. (4-2) provides a distribution of reinforcement that will reasonably control flexural cracking, and is also recommended for controlling cracking due to direct tension. It follows the approach of ACI 318 of emphasizing reinforcing details rather than actual crack width calculations. The equation is based on limiting the crack width \(w\) to 0.013 in. (0.33 mm) using the Gergly-Lutz expression

\[
w = k_w \beta f_{y c} \frac{A}{d_c}
\]

where:

- \(k_w\) = crack width constant
- \(\beta\) = ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement
- \(f_{y c}\) = yield stress of the concrete
- \(A\) = area of the main reinforcement
- \(d_c\) = depth of the concrete

The direct tension limit is based on the following crack width equation for direct tension given in ACI 224.2R

\[
w = 0.138 f_{y c} \frac{A}{d_c} \left[1 + \left(\frac{s}{4d_c}\right)^2\right] = 0.10 f_{y c} \frac{A}{d_c}
\]

for \(s/d_c\) between 1 and 2.

Eq. (4-2) was developed for beams with clear cover typically 2 in. (50 mm) or less, and it will give conservative results with larger concrete cover. To account for this the guide recommends that the clear cover used in calculating \(d_c\) should not exceed 2 in. (50 mm) even though the actual cover is larger.

**A4.5—Snow loads**

Snow loads for ASCE 7 Category III and IV structures are identical except for the importance factor, which is 1.1 and 1.2, respectively.

**A4.5.2.1**—The 0.76 in Eq. (4-3a) is a combined factor that is the rounded product of the following:

(a) flat roof factor: 0.7;
(b) exposure factor: 0.9;
(c) thermal factor: 1.2.

The equation for the slope factor \(C_s\) is based on Fig. (7-2b) of ASCE 7 for a cold, unobstructed slippery surface. The curved roof slope factor is based on Fig. (7-3) of ASCE 7.

**A4.6—Wind forces**

Wind loads for ASCE 7 Category III and IV structures are the same. The wind forces considered here are for rigid structures. The potential for across-wind excitation or flutter should be investigated for tall slender tanks with fundamental periods of 1 sec or greater.

**A4.6.3 Design wind force**—Wind forces acting on the structure are positive and negative pressures acting concurrently. The service load wind force is the sum of the wind pressures times the respective projected areas of portions of the structure. It is assumed that the structure is divided into one or more height zones, and the wind pressure and resultant force are calculated for each zone.

**A4.6.3.1**—The drag coefficients \(C_d\) for cylindrical and doubly curved surfaces are 0.6 and 0.5, respectively, and are from AWWA D100. These values are comparable to the 0.5 to 0.6 obtained from ASCE 7 Table 6-7 for smooth cylindrical shapes with height to diameter ratios in the range of 1 to 7. The use of the upper bound value for cylinders and a lower value for doubly curved surfaces provides a reasonable distinction between the drag forces acting on the two shapes.

**A4.6.3.2**—The minimum recommended wind pressure on a flat surface is 30 lb/ft\(^2\) (1.44 kPa), which has been used successfully for years in the design of elevated tanks. The equation for wind pressure follows ASCE 7 using the combined height and gust response factor \(C_h = K_h G_f\). The height factor \(K_h\) is from the following equations found in the ASCE 7 Commentary

\[
K_h = 2.01 \left(\frac{z}{z_g}\right)^{2/\alpha} \quad \text{for} \quad z_{15} \leq z \leq z_g
\]

\[
K_h = 2.01 \left(\frac{z}{z_{15}}\right)^{2/\alpha} \quad \text{for} \quad z < z_{15}
\]

where

- \(z_{15} = 15\) ft (4.6 m)

**Table A4.6.3.2—Values of \(\alpha\) and \(z_g\)**

<table>
<thead>
<tr>
<th>Exposure category</th>
<th>(\alpha)</th>
<th>(z_g) ft (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>7.0</td>
<td>1200 (370)</td>
</tr>
<tr>
<td>C</td>
<td>9.5</td>
<td>900 (270)</td>
</tr>
<tr>
<td>D</td>
<td>11.5</td>
<td>700 (210)</td>
</tr>
</tbody>
</table>

The gust factor \(G_f\) conforms to ASCE 7 for rigid structures and is equal to 0.8 for structures in Exposure B, and 0.85 in Exposures C and D.

**A4.7—Seismic forces**

**A4.7.1 General**—Seismic loads for ASCE 7 Category III and IV structures in this document are the same.
A4.7.1.3—Experience with concrete-pedestal elevated water-storage tanks is generally in regions where the peak effective ground acceleration is 0.20 g or less, and elastic response is expected. This document provides design procedures for sites to 0.40 g peak effective ground acceleration. Designers are cautioned to carefully evaluate structural response, strength, and requirements for inelastic behavior in higher seismic regions.

A4.7.2—Design seismic force

The minimum seismic forces prescribed in this document are factored loads intended to be used with the strength design load combinations of Section 4.2.3.

Elevated tanks covered by this document behave basically as single-degree-of-freedom systems that respond primarily to the fundamental frequency of vibration, and generally do not warrant sophisticated analysis techniques for determining seismic forces.

Alternative procedures that may be used for analysis include:
(a) Modal analysis using solution to the equations of motion;
(b) Modal analysis using the response spectrum technique;
(c) Finite element analysis using modal analysis or direct integration method.

A4.7.4—Seismic coefficients

A4.7.4.1—The effective peak acceleration \(A_e\) is a coefficient representing ground motion at a period of about 0.1 to 0.5 second. The effective peak velocity-related acceleration coefficient \(A_v\) is a coefficient representing ground motion at a period of about 1.0 second. These coefficients can be considered as normalizing factors for construction of smoothed elastic response spectra of normal duration. The coefficients are related to peak ground acceleration and peak ground velocity but are not necessarily the same or even proportional to peak acceleration and velocity (see NEHRP Commentary).

A4.7.4.3—The response modification coefficient \(R = 2.0\) is in accordance with ASCE 7 Table 9.2.7.5 for non-building, inverted-pendulum structures.

A4.7.5—Structure period

A4.7.5.1—The fundamental period of vibration \(T\) should be determined using established methods of mechanics, assuming the concrete support wall remains elastic during the vibration. The following formula based on Rayleigh’s method (see NEHRP Commentary) is commonly used:

\[
T = 2\pi \sqrt{\frac{\Sigma (w_i\delta_i^2)}{\Sigma g i\Sigma (F_i\delta_i)}}
\]

where \(\delta_i\) = static elastic deflection of the structure at level \(i\) due to forces \(F_i\).

The distribution of \(F_i\) should be approximately in accordance with Section 4.7.7. The lumped-mass loads \(w_i\) may be substituted for the applied lateral forces \(F_i\) without significant loss of accuracy.

A4.7.5.2—The single-mass approximation assumes a cantilever of uniform stiffness with the effective structure mass located at the centroid of the stored water. This is a reasonable approximation for an elevated water tank where the water weight is typically 80 percent of the total structure weight.

The structure lateral stiffness \(k_c\) is determined from the deflection of the concrete support structure acting as a cantilever beam of length \(l_{cg}\) subjected to a concentrated end load. The flexural stiffness for this condition is

\[
k_c = \frac{3E I_c}{l_{cg}^3}
\]

The modulus of elasticity of concrete \(E\) is determined in accordance with ACI 318, and \(I_c\) is the moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement. The use of uncracked section properties to determine stiffness is consistent with ASCE 7 where nonlinear seismic coefficients are used with the elastic structure response.

For this approximation to be acceptable, the relative stiffness of the tank cone should be within 50 percent of the relative stiffness of the concrete support wall.

A4.7.7 Force distribution—Eq. (4-10b) is the seismic force distribution prescribed in ASCE 7. Eq. (4-10a) is a simplification that considers the seismic force distribution to be proportional to the vertical distribution of the structure’s weight. The results differ only slightly when most of the structure mass is contained in the stored water, which is the case for most concrete-pedestal tanks. Where the dead load exceeds approximately 25 percent of the total weight, the Eq. (4-10b) should be used. The simplest and most conservative approach is to consider the entire structure mass located at a single level, the centroid of the stored water. An analysis that considers the individual mass of stored water, steel tank, tank floor, and support wall is usually sufficient for evaluating lateral seismic forces.

A4.7.9 Overturning moment—Eq. (4-13b) assigns half the calculated bending moment at the base to the top of the structure as required by ASCE 7 for inverted pendulum structures. For ease of calculation, the top of the structure is chosen as the centroid of the stored water, which is determined in the course of calculating the structure’s period.

A4.7.11—Other effects

A4.7.11.1—ASCE 7 requires the inclusion of torsional moment caused by an assumed displacement of the mass from its actual location by a distance equal to 5 percent of the structure’s dimension perpendicular to the direction of the applied forces. This is equivalent to increasing the shear stress in the wall by 5 percent at sections where there are no openings. Design of the wall is rarely controlled by horizontal shear stress, and in order to simplify calculations the torsional moment may be neglected when it is less than 5 percent of the shear strength. In structures with large openings that are subjected to high seismic loads the torsional effects may be significant, and should be included.

A4.7.11.2—Moments caused by P-delta effects will be extremely small because the relatively large bending stiffness of the concrete support walls limits lateral deflections to being negligibly small. The axial load on the wall cylinder is relatively concentric.

A4.7.11.3—Anchorage between the steel tank and the concrete support is checked for seismic loads assuming elastic behavior (design seismic load multiplied by seismic
response factor $R$ of 2.0). This is to preclude a possible connection failure during a seismic event.

**A4.8—Support wall**

**A4.8.1 General**—The intent of this section is to not restrict analysis and design methods. Designs based on analysis using finite elements or finite difference solutions are permitted.

**A4.8.2—Details of wall and reinforcement**

The specified compressive strength of concrete is limited to 6000 psi (41 MPa). This is based on current design methods and construction procedures, and is not intended to exclude the use of higher strength concrete. The design and construction problems associated with thin concrete elements must be addressed by the user when higher strength concrete results in thin sections.

**A4.8.3—Vertical load capacity**

**A4.8.3.2**—In this document, the equation for nominal axial strength has the same form as the ACI 318 empirical design equation for walls. Both have a strength reduction term and a slenderness term.

The value of the strength reduction factor $C_w$ used for circular walls is 0.55, the same as that used in ACI 318 for straight walls. Straight walls are sensitive to eccentricity of loading, and ACI 318 considers that walls may be loaded within the middle third (that is, $e < h/6$) and still be designed by the empirical method using $C_w$ equal to 0.55 for circular walls. The centroid of loading will tend to be at the wall centerline at locations remote from section changes, geometric imperfections and other disturbing influences, and a higher value $C_w$ could be used in these regions. However, to account for accidental eccentricity and to provide a conservative design, $C_w$ is taken as constant for the entire wall height.

There may be a reduction in the wall axial strength at high slenderness ratios. The slenderness reduction factor $b_w$ is obtained by equating the classical elastic buckling strength of a cylinder

$$\sigma_{cr} = \gamma C_c E (2 h/d_w) \beta_w f'c,$$

where

- $C_c = 0.59$, for Poisson’s ratio of 0.2;
- $E = 1,800,000$ psi (12,400 MPa), approximation of the long-term modulus of elasticity taken as one-half the short-term modulus for 4000 psi (28 MPa) concrete;
- $\gamma = 1/6.6$, reduction factor;
- $f'c = 4000$ psi (28 MPa), specified compressive strength of concrete;

Substituting and rearranging results in $\beta_w = 80 (h/d_w)$.

**A4.8.3.4**—The bending in the support wall resulting from radial rotation of flexible raft or eccentrically loaded annular ring foundations can be significant, and may control the design at the base of the wall in these situations.

**A4.8.4—Circumferential bending**

**A4.8.4.1**—It is not necessary to add circumferential wind effects to any other loads or effects in determining the requirements for horizontal reinforcement.

**A4.8.4.2**—The equation for circumferential bending moment uses a moment coefficient of 0.052 that was determined from an analysis of a ring subjected to the wind pres-
A4.8.6.8—The equation for nominal shear strength \( V_u \) is a combined form of Eq. (21-6) and (21-7) from Chapter 21 of ACI 318 for shear walls and diaphragms. High in-plane shear forces usually only occur with seismic forces, and the use of this equation results in a design compatible with ACI 318 seismic requirements.

The coefficient \( \alpha_c \) in Eq. (4-23) is from Section 21.6.5.3 of ACI 318. In in.-lb units, the linear portion having values between 2.0 and 3.0 can be written in equation form as \( \alpha_c = 6 - 2(h/l) \). Substituting \( M_u / V_u \) for \( h \) and 0.78\( d_w \) for \( l \) gives \( \alpha_c = 6 - 2.56(M_u / (V_u d_w)) \). In Eq. (4-23) the 2.56 coefficient is rounded to 2.5. In the SI system, the linear portion varies between 1/6 and 1/4, and results in \( \alpha_c = 0.5 - 0.21M_u / (V_u d_w) \).

A4.8.6.9—The location for determining nominal shear strength is the lower of the mid-height of the largest opening or a distance equal to one-half the effective shear wall width above the base. The second criterion is consistent with ACI 318, and the first ensures that shear across openings is checked.

A4.8.6.10—The minimum reinforcement requirements in Table 4.8.2 conform to the requirements of Chapter 21 of ACI 318 for shear walls. The additional requirements of Section 4.8.6.10 apply only to regions of high seismic risk. High seismic risk regions are generally defined as regions where \( A_r \) is greater than 0.20, or seismic Zones 3 and 4 in the Uniform Building Code or BOCA National Building Code.

A4.9—Tank floors

A4.9.1 General—Concrete tank floors covered by this section are limited to uniform-thickness flat slabs and concrete domes. Other configurations may be used, but they should have the same strength and behavior as the tank floors covered by Sections 4.9.2 and 4.9.3.

Access tubes used as roof support columns can transfer significant axial loads to the concrete floor, and may govern its design.

A4.9.2 Flat slab floors—Flat slab floors are designed as two-way slabs or plates. In addition to meeting ACI 318 strength requirements for shear and flexure, the slab should be checked for serviceability. Thin slabs with adequate strength may be too flexible for the attached cone and wall to function properly. Excessive rotation can cause premature buckling of the cone, and cracking in the wall. The minimum flexural reinforcement requirement (Section 4.3.3) is intended to ensure adequate stiffness at cracked sections. At other sections, temperature and shrinkage reinforcement equal to 0.002 times the gross concrete area is required.

A4.9.3 Dome floors—Concrete domes act as membranes having in-plane compression forces, except near the inner and outer edges where edge forces and deformation incompatibilities with attached elements occur. The shear and moment in these edge regions should be considered in the design. The minimum flexural reinforcement requirement (Section 4.3.3) is intended to ensure adequate stiffness at cracked sections. At other sections a minimum steel ratio of 0.002 in each face is recommended. For most domes this is equivalent to No. 4 (13) or No. 5 (16) bars at 12 in. (300 mm) and provides an allowance for loads, such as construction loads, that may not be accounted for directly in the design.

The minimum thickness given by Eq. (4-24) is based on limiting the service-load membrane compression stress to between 500 and 600 psi (3.4 and 4.1 MPa). The resulting radius to thickness ratio is in the range of 50 to 100 for most tank geometries. When the radius to thickness ratio exceeds 100, shell buckling should be considered by another method.

A4.10—Concrete to tank interface

A4.10.2 General design considerations—The conditions at the interface can be sensitive to tank and support wall proportions, and to the initial assumptions in ways that are not intuitively obvious. For this reason, full analysis is required.

A series of analyses of the interface area for a particular tank configuration is usually required. This series should investigate the plausible range of material properties, volume change effects (creep and shrinkage), construction tolerances, variable water load, and environmental loads.

It is advisable to keep direct tension stress in circumferential reinforcement less than 5000 to 10,000 psi (35 to 70 MPa) to prevent excessive hoop deformation and cracking in the interface region.

A4.10.3 Dome floors—The wall, dome, and tank cone are shell elements that resist load by membrane action. Where they are connected together by a ringbeam, or similar element, the shell element boundary conditions are not compatible with membrane action and out-of-plane forces and deformations result. Appropriate analytical means (for example finite element analysis) should be used to determine...
the stress in connecting elements in the vicinity of the ringbeam. Where the analysis shows a net tensile strain in the ringbeam, the lower portion of the steel cone and its steel connecting elements may be included as elements resisting tension forces. The wall and dome should be designed for shear, direct tension and flexural tension caused by interaction effects in the vicinity of the ringbeam.

A4.10.4 Slab floors—Shrinkage will affect the forces and stresses arising from the incompatibility of strains between the tank cone and the slab. Shrinkage will also cause bending moments in the top of the wall, for which reinforcement should be provided in the wall both vertically and circumferentially some distance below the top of the wall.

A4.10.5—Suspension of slabs during their service life.

A4.10.5.1—This design has no concrete dome or slab to maintain circularity and is sensitive to construction tolerances that can cause additional load due to misalignment.

A4.10.5.3 and A4.10.5.4—The requirements for concrete elements supporting base plates reflect the difficulties associated with centering annular base plates on narrow concrete supports, the effect of eccentric loads on load-carrying capacity, and the potential for edge spalling. Experience with grouted base plates supported directly on the support wall has shown that regular checking and adjustment of diameter and curvature are necessary to position the center of the base plate and skirt at the center of the wall. Where no special measures are implemented to ensure fit of the steel tank to the concrete construction, a ringbeam is recommended. Constructors that build without a ringbeam generally use lasers to control alignment and shape during wall construction, and check the as-built condition to ensure that the base plate will fit on the wall within the specified tolerances. A combined inside and outside edge distance not less than 6 in. (150 mm) is required to ensure that the base plate will fit properly on the wall. A smaller edge distance may be used when experience with the form system, steel fabrication, and construction tolerance controls result in fit of the base plate to the wall construction within the tolerances specified.

A4.10.5.5—The minimum recommended base plate width of 6 in. (150 mm) is the minimum required for using anchor bolts, and provides for grout side cover of shim stacks that are generally not removed. The grout bearing strength for factored loads is conservatively limited to 2000 psi (14 MPa) because of the difficulty of inspection and maintenance of this detail.

A4.10.5.6—The grout that transfers the steel tank load to the concrete support structure is a key structural element that is not readily accessible for inspection and maintenance, and is often neglected in specifications and construction. Only high-quality, durable materials should be used, and their installation should be carefully supervised to ensure that the work complies with specifications and manufacturer’s recommendations.

A4.10.6 Reinforcement details—Minimum radial and circumferential reinforcement of 0.25 percent should be provided for temperature and shrinkage during construction or when the tank is empty, even at sections that are in compression during their service life.

A4.11—Foundation design

A4.11.3 Overturning—The location of the centroid of the resisting load used to calculate the stability ratio depends on the rigidity of the foundation used. The centroid of the resisting load may be taken near the edge of raft foundations, but may be near or inside the support structure wall when an annular ring foundation is used.

A4.11.4.1—The ratio of foundation outside diameter to mean support wall diameter will not exceed approximately 1.45 for an annular ring foundation [Fig. 4.11.1(a)] designed as a one-way strip where torsional effects and biaxial bending are not considered. Where bearing capacity or settlement limits require a foundation with a significantly larger contact area, a raft or a deep foundation should be considered, or torsional effects and biaxial bending must be included in designing the annular ring foundation.

A4.11.5.3—When lateral loads are resisted by piles or piers, significant in-plane shear and moment may exist in annular ring pile or pier caps. For narrow annular ring caps of large diameter, it may be necessary to provide a diaphragm to distribute the load to the piles or drilled piers.

A4.11.6.3—A check at service loads is required because a load factor of less than 1.7 for water may result in high service load stress in the foundation reinforcement. The 30,000 psi (205 MPa) limit on tension reinforcement stress at service loads provides a reasonable upper limit to prevent excessive deflection and cracking of concrete without extensive computational effort. Alternatively, Eq. (4-2) can be used for crack control.

A4.12—Geotechnical recommendations

A4.12.1.2—Structure configuration, loads, and minimum requirements for the geotechnical investigation should be provided to the design professional responsible for the geotechnical investigation. Local experience is a valuable asset in evaluating potential geotechnical and geological conditions that may affect the foundation design and construction. This experience may also provide for a more efficient investigation. Field investigation and laboratory testing should conform to ASTM specifications and recognized procedures.

The geotechnical report should document the results of the field investigation, laboratory testing, analysis, and provide recommendations. The report should contain the following:

(a) Geology of the site, boring logs and classification of soils;
(b) Summary of field and laboratory testing;
(c) Ground water encountered, and the depth at which buoyancy should be considered;
(d) Seismic soil type classification, and assessment of soil liquefaction potential;
(e) Foundation soil stiffness for dynamic analysis, when required;
(f) Bearing capacity and depth of bearing stratum for shallow foundation;
(g) Types of deep foundations that may be used; probable bearing stratum; and expected capacity;
(h) Lateral load capacity of deep foundations;
(i) Tension uplift capacity of deep foundations;
(j) Unit weight of compacted backfill soils;
(k) Construction requirements, including: expected stable excavation slopes, dewatering requirements, use of site soils for backfill, and compaction requirements.

A4.12.2 Foundation depth—Maps of frost penetration depth are available from the U.S. Weather Bureau, or can be found in model building codes or their commentary.

A4.12.3 Settlement limits—Concrete-pedestal tanks and their foundations are relatively rigid structures that can undergo significant total settlement without distress. Settlement of deep foundations is usually smaller than that of shallow foundations, and the smaller settlement limit reflects the expected behavior. The effects of foundation movement relative to slabs and piping should be considered and provided for by properly designed connection details.

Tilting of the structure caused by differential settlement across the foundation width causes secondary overturning moments, and the structural effects of this are accounted for in the design of the superstructure and foundation by the eccentricity load term \( G \). A minimum assumed tilt of 1/800 is included in the design through Eq. (4-1a). Larger differential tilt is permitted when included in Eq. (4-1b).

A4.12.4 Shallow foundations—A global factor of safety of 3.0 is used for sizing shallow foundations using allowable stress design. Where settlement controls the design, the factor of safety is even larger. Analytical methods for determining the maximum bearing pressure can be found in references on foundation design.

Ultimate strength design may also be used to determine required bearing area using the following equation

\[ q_u = \Phi_s q_r \]

The factored soil bearing pressure \( q_u \) is calculated from loads in Section 4.2.3. The shallow foundation performance factor \( \Phi_s \) is 0.5.

The shallow foundation performance factor \( \Phi_s \) used with ultimate strength design provides a global factor of safety of 3.0 when used with the factored load combinations of Section 4.2.3 whose average load factor is approximately 1.5. The ultimate bearing capacity \( q_r \) is the smaller of: 3.0 times the allowable bearing capacity for settlement, or the ultimate bearing capacity of the soil determined in Section 4.12.4.1.

A4.12.5 Deep foundations—A variable global factor of safety that depends on the method of determining the ultimate capacity is used with deep foundations. Where settlement controls the design, the factor of safety will be even larger. Deep foundation elements, such as drilled piers, whose capacity is based on calculations use a minimum factor of safety of 3.0, the same as for shallow foundations. Where static load testing is used the safety factor is reduced to 2.0. Analytical methods for determining the maximum pile or pier load can be found in references on foundation design.

Ultimate strength design may also be used to determine the required number of drilled piers or piles using the following equation

\[ Q_u = \Phi_p Q_r \]

The factored pile or pier load \( Q_u \) is calculated from loads in Section 4.2.3. The deep foundation performance factor \( \Phi_p \) is from Table A4.12.5.

<table>
<thead>
<tr>
<th>Table A4.12.5—Performance factor for deep foundations</th>
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<tr>
<td>Ultimate Capacity in Accordance with Section</td>
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<td>4.12.5.1(a)</td>
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<td>4.12.5.1(c)</td>
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<td>4.12.5.1(d)</td>
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The deep foundation performance factor \( \Phi_p \) used with ultimate strength design provides the same global factor of safety as listed in Table A4.12.5 when used with the factored load combinations of Section 4.2.3 where the average load factor is approximately 1.5.

A4.12.6 Seismic requirements—Relatively fine-grained soils in a relatively loose state of compaction, and in a saturated or submerged condition are subject to liquefaction during earthquake excitation. Where these soils occur, the suitability of a site for a concrete-pedestal elevated water-storage tank should be carefully evaluated. Any special precautions that are required in the design should be identified prior to design.

CHAPTER 5—APPURTENANCES AND ACCESSORIES COMMENTARY

A5.4—Tank Access

A5.4.2 Ladders—The following details are commonly used for ladders.

(a) Side rails are a minimum \( \frac{3}{4} \) in. (10 mm) by 2 in. (50 mm) with a 16 in. (410 mm) clear spacing. Rungs are a minimum \( \frac{3}{4} \) in. (20 mm) round or square, spaced at 12 in. (300 mm) centers. The surface should be knurled, dimpled, coated with skid-resistant material, or otherwise treated to minimize slipping.

(b) At platforms or landings the ladder extends a minimum of 48 in. (1.2 m) above the platform. Ladders are secured to the adjacent structure by brackets located at intervals not exceeding 10 ft (3 m). Brackets have sufficient length to provide a minimum distance of 7 in. (180 mm) from the center of rung to the nearest permanent object behind the ladder.

Where cages are provided, ladders should be offset at landing platforms. The maximum interval between platforms should not exceed 30 ft (9 m). Cages should start between 7 and 8 ft (2.1 and 2.4 m) above the base of the ladder and should extend a minimum of 48 in. (1.2 m) above the offset landing platform.

A5.4.5 and A5.4.6 Steel tank roof openings and floor manhole—Commonly used opening sizes for access to the tank interior are given. Openings used only for personnel access should have a least dimension of 24 in. (610 mm) or larger. Larger openings may be required for painter’s equipment or other interior maintenance items. At least one opening should be of sufficient size to accommodate the largest anticipated equipment. A tank floor manhole is not required for operation and maintenance of the tank, but is considered an appurtenance for safety reasons in that it provides a means of egress other than the roof manholes. Furthermore, it is a
practical access opening during construction and out-of-service maintenance.

A5.6—Above-ground piping

A5.6.2.6—Typically inlet safety protection is provided for pipes of 18 in. (460 mm) diameter and greater, but some jurisdictions may require inlet protection for pipes as small as 8 in. (200 mm) in diameter.

A5.6.3 Overflow—An overflow is a protection device intended to prevent over-filling and possible overloading of the tank. An overflowing tank should be considered an emergency condition. The condition causing the tank to overflow should be promptly determined and rectified by the operator.

A5.8—Interior floors

A5.8.2—Slabs on grade

A5.8.2.1—It is assumed that if door openings are less than 8 ft (2.4 m) wide, that the floor slab will only be subjected to foot traffic and occasional light vehicle traffic. Truck loading should be expected for wider doors.

The optimum concrete mix has the maximum flexural strength with the least mixing water to minimize shrinkage. Concrete with a specified compressive strength in the range of 3500 to 4000 psi (24 to 28 MPa) and a water-cementitious material ratio less than 0.45 can be expected to provide reasonable performance.

The 5 in. (125 mm) minimum thickness is recommended when only light vehicle traffic is expected, and a 4 in. (100 mm) thickness may be used when only foot traffic is expected. The 6 in. (150 mm) minimum thickness recommended where truck doors are furnished is adequate for 15 ton (13.6 tonne) axle loads for most subgrade soils.

The minimum reinforcement requirements in Table 5.8.2 are shrinkage and temperature steel requirements in accordance with ACI 318.

A5.8.2.2—The inside diameter of most concrete pedestals is less than 70 ft (21 m), and the minimum reinforcement in Table 5.8.2 is adequate for controlling cracking for slab widths to this width. This is based on the subgrade drag equation in ACI 360 using a friction coefficient of 2.0 and an allowable stress in reinforcement of 40,000 psi (276 MPa). Continuous reinforcement with contraction joints is commonly used.

A5.8.2.5—Minimum compaction of 95 percent modified Proctor density is recommended for backfill supporting a slab-on-grade subject to vehicle traffic. Otherwise, backfill should be compacted to a minimum of 90 percent modified Proctor density.

A5.8.3 Intermediate floors—Various structural configurations are used for above grade floors. The simplest is a flat slab supported by the wall, which may be used for small diameter walls. For larger spans, flat slabs with intermediate columns, and concrete or steel beams supporting a concrete slab are generally used. Care must be taken that the loads from the beam end reactions are adequately transferred to the supporting wall, and are accounted for in the design.

Commentary recommended references—The following documents with their serial designation are cited in the Commentary only. Other references are listed in Chapter 6.

American Concrete Institute

ACI 201.2R Guide to Durable Concrete
ACI 212.3R Chemical Admixtures for Concrete
ACI 212.4R Guide for the Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete
ACI 224.2R Cracking of Concrete Members in Direct Tension
ACI 301 Standard Specification for Structural Concrete
ACI 303R Guide to Cast-in-Place Architectural Concrete Practice
ACI 307 Standard Practice for the Design and Construction of Cast-in-Place Reinforced Concrete Chimneys
ACI 334.2R Reinforced Concrete Cooling Tower Shells—Practice and Commentary

American Association of State Highway and Transportation Officials

Standard Specifications for Highway Bridges

American Water Works Association

D100 Standard for Welded Steel Tanks for Water Storage

The above publications may be obtained from the following organizations:

American Association of State Highway and Transportation Officials
444 North Capitol Street NW, Suite 225
Washington, D.C. 20001

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

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

Commentary cited references—The following document is cited in the Commentary only. Other cited references are listed in Chapter 6.