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**ANSI/AWWA D107-16**  
(Revision of ANSI/AWWA D107-10)

**AWWA Standard**



# Composite Elevated Tanks for Water Storage

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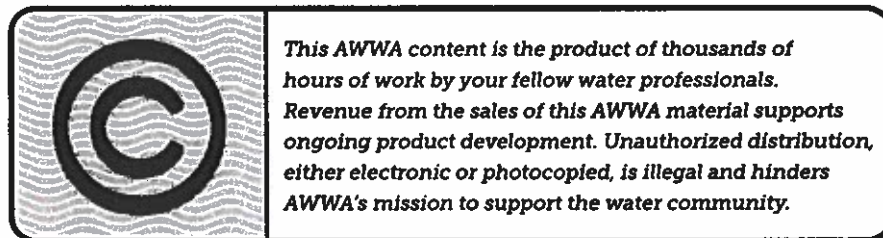
## AWWA Standard

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† Alternate

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# Foreword

*This foreword is for information only and is not a part of ANSI/AWWA D107.*

## I. Introduction.

I.A. *Background.* A composite elevated water tank is composed of a welded steel tank for watertight containment, a single pedestal concrete support structure, a foundation, and accessories.

The AWWA Standards Committee on Composite Elevated Tanks was formed to prepare a standard for the design, construction, inspection, and testing of composite elevated tanks. ACI 371R, Guide for the Analysis, Design, and Construction of Elevated Concrete and Composite Steel-Concrete Water Storage Tanks,<sup>†</sup> and ANSI/AWWA D100, Welded Carbon Steel Tanks for Water Storage, are used as source documents.

Work covered by this standard is usually procured under a design-build contract. It is intended that ANSI/AWWA D107 be used as a reference standard in project documents prepared by purchasers and engineers specifying composite elevated water tanks.

I.B. *History.* The AWWA Standards Committee on Composite Elevated Tanks was formed in 1992 to prepare a standard for these structures. The first edition of ANSI/AWWA D107 was approved by the AWWA Board of Directors on Jan. 17, 2010. This edition was approved on Jan. 16, 2016.

I.C. *Acceptance.* In May 1985, the US Environmental Protection Agency (USEPA) entered into a cooperative agreement with a consortium led by NSF International (NSF) to develop voluntary third-party consensus standards and a certification program for direct and indirect drinking water additives. Other members of the original consortium included the Water Research Foundation (formerly AwwaRF) and the Conference of State Health and Environmental Managers (COSHEM). The American Water Works Association (AWWA) and the Association of State Drinking Water Administrators (ASDWA) joined later.

In the United States, authority to regulate products for use in, or in contact with, drinking water rests with individual states.<sup>‡</sup> Local agencies may choose to impose requirements more stringent than those required by the state. To evaluate the health effects of products and drinking water additives from such products, state and local agencies may use various references, including

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\* American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036.

† American Concrete Institute (ACI), 38800 Country Club Drive, Farmington Hills, MI 48331.

‡ Persons outside the United States should contact the appropriate authority having jurisdiction.

1. Specific policies of the state or local agency.
2. Two standards developed under the direction of NSF\*: NSF/ANSI 60, *Drinking Water Treatment Chemicals—Health Effects*, and NSF/ANSI 61, *Drinking Water System Components—Health Effects*.
3. Other references, including AWWA standards, *Food Chemicals Codex*, *Water Chemicals Codex*,<sup>†</sup> and other standards considered appropriate by the state or local agency.

Various certification organizations may be involved in certifying products in accordance with NSF/ANSI 61. Individual states or local agencies have authority to accept or accredit certification organizations within their jurisdictions. Accreditation of certification organizations may vary from jurisdiction to jurisdiction.

Annex A, “Toxicology Review and Evaluation Procedures,” to NSF/ANSI 61 does not stipulate a maximum allowable level (MAL) of a contaminant for substances not regulated by a USEPA final maximum contaminant level (MCL). The MALs of an unspecified list of “unregulated contaminants” are based on toxicity testing guidelines (noncarcinogens) and risk characterization methodology (carcinogens). Use of Annex A procedures may not always be identical, depending on the certifier.

ANSI/AWWA D107 does not address additives requirements. Users of this standard should consult the appropriate state or local agency having jurisdiction in order to

1. Determine additives requirements, including applicable standards.
2. Determine the status of certifications by parties offering to certify products for contact with, or treatment of, drinking water.
3. Determine current information on product certification.

## II. Special Issues.

II.A. *Applicable Building Codes.* Building codes may not have specific provisions for nonbuilding structures such as composite elevated tanks. The purchaser should make a determination as to the intent of the requirements of the applicable building codes and should specify the extent to which such building codes apply to a project. It is intended that specified building codes govern where they have more stringent requirements than this standard.

II.B. *Personnel Safety Standards.* The personnel safety requirements in this standard are based on OSHA 29 CFR, Part 1910, Occupational Safety and Health Standards, and industry practice. The user in specifying safety-related equipment,

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\* NSF International, 789 North Dixboro Road, Ann Arbor, MI 48105.

† Both publications available from National Academy of Sciences, 500 Fifth Street, NW, Washington, DC 20001.

details, and design criteria should make a determination that the requirements of this standard are in compliance with current OSHA requirements and applicable building code requirements.

II.C. *Special Loading Conditions.* The purchaser should specify design requirements for service not covered by this standard, such as loads associated with surge or process tanks, or local environmental conditions more severe than those required by this standard.

II.D. *Professional Engineer (PE) Certification.* It is recommended that the engineer responsible for design be licensed in the jurisdiction where the composite elevated tank is to be constructed. The design and construction drawings should be sealed.

II.E. *Inspection and Maintenance.* Composite elevated tanks designed and constructed in accordance with the requirements of this standard can be expected to be durable structures. Steel tank construction has a satisfactory history when properly maintained, and concrete that conforms to this standard will meet the requirements for durable concrete as defined in ACI 201.2R, Guide to Durable Concrete.

An inspection of the structure and accessories is recommended approximately one year after completion. Thereafter, periodic inspection and maintenance should be performed.

II.F. *Cold Climates.* The formation of ice in the tank may result in damage to the tank or components such as piping or interior ladders. Design for this condition is beyond the scope of this standard. Reference to ice damage is to improper operation rather than an endorsement of an icing condition. The tank should be heated, insulated, or operated in a manner to prevent this condition. Water replacement, circulation, or wasting of water are operational techniques that may be used.

II.G. *Use of Interior Space.* The interior of the support structure is frequently used for mechanical rooms, storage, and parking. This space may be occupied when constructed in accordance with local building code requirements. Typical uses include fire stations and offices. Intermediate floors may be constructed to provide additional space.

II.H. *Aesthetics.* Aesthetics should be considered when planning a composite elevated tank. Minimum architectural features and finishing of exterior concrete surfaces are provided for in this standard. Painting decorative logos or color schemes on the steel tank may enhance the appearance of the structure.

II.I. *Structural Evaluation.* If construction does not meet tolerance, material, or other structural requirements of this standard, an evaluation may be performed

by the responsible design professional. When structural capacity is not compromised, repair or replacement may not be required unless other factors, such as lack-of-fit, aesthetics, or durability, require remedial action.

### **III. Use of This Standard.**

#### **III.A. General.**

III.A.1 Disclaimer. AWWA has no responsibility for the suitability or compatibility of the provisions of this standard to any intended application by any user. Accordingly, it is the responsibility of the user of this AWWA standard to determine that the provisions described in this standard are suitable for use in the particular application being considered.

III.A.2 Minimum Requirements. ANSI/AWWA D107 is based on the collective knowledge of purchasers, consulting engineers, and constructors of composite elevated tanks. A composite elevated tank is considered to be in compliance with ANSI/AWWA D107 when the minimum requirements of this standard have been met. The purchaser may specify more stringent requirements.

III.A.3 Contract Responsibility. Procurement of composite elevated tanks is usually by design-build contracts using proprietary designs and methods of construction. The purchaser should provide project documents that describe the work to be performed.

The purchaser typically is responsible for the following:

1. A site on which the composite elevated tank is to be constructed, which should be of sufficient size to permit construction using customary methods.
2. A suitable right-of-way from the nearest public road to the construction site.
3. A geotechnical investigation, including recommendations for foundation type and associated design criteria.
4. A Federal Aviation Administration (FAA) determination with regard to siting, height, marking, and lighting requirements.
5. Water at sufficient pressure for testing and facilities for disposal after testing.
6. Purchaser-furnished materials that are to be installed by the constructor.

The constructor typically is responsible for the following:

1. Designs, drawings, and specifications for the composite elevated tank.
2. All labor, materials, equipment, supplies, and testing as required by this standard.
3. Warranting the structure against defects in material and workmanship for a period of one year from date of completion.
4. Any additional work specified in the project documents.

### III.B. *Specifying ANSI/AWWA D107.*

III.B.1 Reference Standard. ANSI/AWWA D107 is a reference standard that a purchaser may cite in project documents, together with supplementary requirements for a specific project. A statement such as the following may be used to incorporate ANSI/AWWA D107 into the project documents:

“Design and construction of the composite elevated tank shall comply with the requirements of ANSI/AWWA D107, Composite Elevated Tanks for Water Storage, published by the American Water Works Association, Denver, Colorado, except as modified by the requirements of these contract documents.”

III.B.2 Information Required From Purchaser. Incorporating this standard by reference in a project specification requires that the purchaser provide the information listed in Sec. 1.3. ANSI/AWWA D107 has options that the purchaser may specify, and Section B.2 of appendix B, Table B.1, contains a comprehensive list of these options and corresponding defaults.

III.C. *Modification of Standard.* Any modification of the provisions, definitions, or terminology in this standard must be provided in the project specifications.

**IV. Major Revisions.** The major revisions in this edition of the standard include the following:

1. The notation and definition for maximum considered earthquake  $MCE_R$  have been revised to conform to the latest edition of ASCE\* 7.
2. Seismic maps are now cited by reference to ASCE 7-10 rather than being incorporated into the standard.
3. Former Table 3, Site Classifications, has been moved to appendix A and has been renumbered as Table A.2.
4. Standard Sec. 4.2.7.8, Seismic Design Category, and appendix Sec. A.4.2.7.8, Seismic Load, have been added.
5. ASCE 7-10 wind provisions have been incorporated into the standard.
6. The gust effect factor  $G_f$  has been revised.
7. The wind map (Figure 2 of the 2010 edition) has been deleted and is now incorporated by reference to ASCE 7-10.
8. Former Tables 2a and 2b on design wind pressure have been revised and moved to appendix A as Tables A.1a and A.1b.
9. Former Table 14, Minimum Safety Factors (now Table 12), has been revised; ASTM D4945 has been added to the dynamic testing provisions.

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\* American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, VA 20191.

10. Standard Sec. 6.3.3.8.1 and appendix Sec. A.6.3.3.8.1 have been updated to reflect current practice, similar to the provisions described in ACI 371R-08.

11. Standard Sec. 7.3.3.2 and appendix Sec. A.7.3.3 have been updated, whereby allowable compression stress values are now referenced to IBC 2012 rather than being provided in the standard.

12. Appendix Section A.2 has been added to identify specific revision years for ASCE 7 and ACI 318.

13. Determination of meridional compression strength  $F_L$  by method 2 (ANSI/AWWA D107-10, Sec. 5.3.5.3) has been eliminated. Method 3 in ANSI/AWWA D107-10 has been renamed as method 2 in this edition. Section numbers have been revised accordingly.

**V. Comments.** If you have any comments or questions about this standard, please call AWWA Engineering and Technical Services at 303.794.7711, FAX at 303.795.7603; write to the department at 6666 West Quincy Avenue, Denver, CO 80235-3098; or email at [standards@awwa.org](mailto:standards@awwa.org).



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# Composite Elevated Tanks for Water Storage

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## SECTION 1: GENERAL

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### **Sec. 1.1 Scope**

This standard describes the design, construction, inspection, and testing of composite elevated tanks that use a welded steel tank for watertight containment and a single pedestal concrete support structure. Requirements for the steel tank, concrete support structure, foundation, and accessories are included. Site selection and procurement; tank sizing; postcommissioning inspection and maintenance; and the design, operation, and control of the water distribution system that connects to the composite elevated tank are beyond the scope of this standard.

### **Sec. 1.2 Purpose**

The purpose of this standard is to provide minimum requirements for the design, construction, inspection, and testing of composite elevated tanks used for water storage in a water distribution system.

### **Sec. 1.3 Application**

This standard may be referenced in project documents that address the design, construction, inspection, and testing of composite elevated tanks used for water storage. The following information is required to use this standard to specify a composite elevated tank:

1. Project location.
2. Tank capacity.
3. Maximum head range.
4. Elevation of the top capacity line (TCL) or bottom capacity line (BCL).
5. Elevation of finished grade.
6. Maximum fill rate.
7. Size of inlet/outlet pipe.

#### **Sec. 1.4 Drawings, Calculations, and Instructions**

1.4.1 *Drawings.* Construction drawings for the composite elevated tank shall be provided. Drawings shall show all features of the work, including the size and position of all structural components, the required strength or grade of all materials, and construction tolerances. Drawings of concrete elements shall show all details including construction joints, reinforcement, and splices. Drawings of steel components shall show all details of welded joints and other connections. Standard weld symbols as listed in AWS 2.4 shall be used, unless joint details are shown. The codes and standards to which the design conforms, the tank capacity, and the loads used in the design shall also be shown.

1.4.2 *Design basis documentation.* The codes and standards, methods of analysis, design coefficients, and resultant gravity, snow, wind, and seismic loads shall be documented.

1.4.3 *Operating and maintenance instructions.* Unless otherwise specified, operating and maintenance instructions shall be provided. These instructions shall include as-built drawings of the composite elevated tank, manuals and operating instructions for equipment and accessories, and minimum maintenance and inspection instructions.

#### **Sec. 1.5 Quality Assurance**

A quality assurance plan to verify that materials, fabrication, and construction conform to the design requirements shall be prepared. The plan shall include the following:

1. Procedures for exercising control of fabrication and construction.
2. Required inspections and tests.
3. Inspection and test procedures.

#### **Sec. 1.6 Equivalence Between US Customary Units and SI-Metric Units of Nonhomogeneous Equations**

Metric equivalents in the *Système International d'Unités* (SI) of nonhomogeneous equations presented in the standard are as follows:



Equation or Section Number	US Customary Equation	SI Equivalent Metric Equation	Variables	SI Units
Eq. 4-7	$q_s = 0.00256V_b^2$	$q_s = 0.6133V_b^2$	$q_s$ $V_b$	N/m <sup>2</sup> m/sec
Eq. 5-1	$L = \frac{2.575t}{\sqrt{w_r}} \leq 84 \text{ in.}$	$L = \frac{17.8t}{\sqrt{w_r}} \leq 2.13 \text{ m}$	$L$ $t$ $w_r$	m mm N/m <sup>2</sup>
Eq. 6-1	$S = 15 \left( \frac{40,000}{f_s} \right) - 2.5c_c \leq 12 \left( \frac{40,000}{f_s} \right)$	$S = 380 \left( \frac{280}{f_s} \right) - 2.5c_c \leq 300 \left( \frac{280}{f_s} \right)$	$S, c_c$ $F_s$	mm MPa
Eq. 6-8	$\alpha_c = 6 - \frac{2.5M_u}{V_u D_w}$ $2.0 < \alpha_c \leq 3.0$ $D\sqrt{q_z} > 2.5$	$\alpha_c = 0.37 - \frac{0.10M_u}{V_u D_w}$ $0.17 < \alpha_c \leq 0.25$ $D\sqrt{q_z} > 5.3$	$\sqrt{f_c}$ $M_u, V_u, D_w$	MPa consistent units
A.4.2.6.3			$q_z$ $D$	N/m <sup>2</sup> m
<b>Terms including <math>\sqrt{f_c} f_y</math></b>				
6.3.2.2	$3\sqrt{f_c} f_y \geq 200/f_y$	$0.25\sqrt{f_c} f_y \geq 1.4/f_y$	$\sqrt{f_c}$ $f_c f_y$	MPa MPa
6.3.2.3	$5\sqrt{f_c} f_y$	$0.42\sqrt{f_c} f_y$		
<b>Terms including <math>\sqrt{f_c}</math> and <math>\sqrt{f_c} A_{cv}</math></b>				
A.6.3.2.3	$7.5 \sqrt{f_c}$ in psi	$0.62 \sqrt{f_c}$ in MPa	$\sqrt{f_c}$ $f_c'$ $A_{cv}$	MPa MPa mm <sup>2</sup>
A.6.3.3.3	$\sqrt{f_c} A_{cv}$	$0.083\sqrt{f_c} A_{cv}$		
6.3.3.8.2	$2\sqrt{f_c} A_{cv}$	$0.17\sqrt{f_c} A_{cv}$		
A.6.3.3.8.2				
6.3.3.8	$8\sqrt{f_c} A_{cv}$	$0.67\sqrt{f_c} A_{cv}$		

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## SECTION 2: REFERENCES

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The following standards and specifications are incorporated by reference. They form a part of this standard to the extent specified herein. Except as noted, the standard or specification in effect on the effective date of this standard shall be used.

ACI\* 117—Specifications for Tolerances for Concrete Construction and Materials and Commentary.

ACI 306.1—Standard Specification for Cold Weather Concreting.

ACI 318—Building Code Requirements for Structural Concrete and Commentary.

AISC†—Code of Standard Practice for Structural Steel Buildings and Bridges.

ANSI/AISC 360—Specification for Structural Steel Buildings.

ANSI/AWWA C200—Steel Water Pipe, 6 in. (150 mm) and Larger.

ANSI/AWWA C205—Cement-Mortar Protective Lining and Coating for Steel Water Pipe—4 in. (100 mm) and Larger—Shop Applied.

ANSI/AWWA C210—Liquid-Epoxy Coating and Linings for Steel Water Pipe and Fittings.

ANSI/AWWA C213—Fusion-Bonded Epoxy Coating and Linings for Steel Water Pipe and Fittings.

ANSI/AWWA C220—Stainless-Steel Pipe, ½ in. (13 mm) and Larger.

ANSI/AWWA C652—Disinfection of Water-Storage Facilities.

ANSI/AWWA D100—Welded Carbon Steel Tanks for Water Storage.

ANSI/AWWA D102—Coating Steel Water-Storage Tanks.

API‡ Standard 650—Welded Tanks for Oil Storage.

ASCE§ 7—Minimum Design Loads for Buildings and Other Structures.

ASCE 20—Standard Guidelines for the Design and Installation of Pile Foundations.

ASME¶—Boiler and Pressure Vessel Code, Section V—Nondestructive Examination, Article 2.

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\* American Concrete Institute, 38800 Country Club Drive, Farmington Hills, MI 48331.

† American Institute of Steel Construction, 1 East Wacker Drive, Suite 700, Chicago, IL 60601.

‡ American Petroleum Institute, 1220 L Street, NW, Washington, DC 20005.

§ American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, VA 20191.

¶ American Society of Mechanical Engineers, 3 Park Avenue, New York, NY 10016.

ASME—Boiler and Pressure Vessel Code, Section VIII, Division 1—Rules for Construction of Pressure Vessels.

ASME—Boiler and Pressure Vessel Code, Section IX—Welding, Brazing, and Fusing Qualifications.

ASNT\*—Recommended Practice No. SNT-TC-1A: Personnel Qualification and Certification in Nondestructive Testing.

ASTM<sup>†</sup> A6/A6M—Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling.

ASTM A20/A20M—Standard Specification for General Requirements for Steel Plates for Pressure Vessels.

ASTM A36/A36—Standard Specification for Carbon Structural Steel.

ASTM A123/A123M—Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products.

ASTM A435/A435M—Standard Specification for Straight-Beam Ultrasonic Examination of Steel Plates.

ASTM A516/A516M—Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service.

ASTM A537/A537M—Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel.

ASTM A572/A572—Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.

ASTM A573/A573M—Standard Specification for Structural Carbon Steel Plates of Improved Toughness.

ASTM C31/C31M—Standard Practice for Making and Curing Concrete Test Specimens in the Field.

ASTM C33/C33M—Standard Specification for Concrete Aggregates.

ASTM C39/C39M—Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.

ASTM C42/C42M—Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.

ASTM C138/C138M—Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete.

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\* American Society of Nondestructive Testing, P.O. Box 28518, 1711 Arlingate Lane, Columbus, OH 43228.

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

ASTM C143/C143M—Standard Test Method for Slump of Hydraulic-Cement Concrete.

ASTM C172/C172M—Standard Practice for Sampling Freshly Mixed Concrete.

ASTM C173/C173M—Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method.

ASTM C231/C231M—Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method.

ASTM C309—Standard Specification for Liquid Membrane-Forming Compounds for Curing Concrete.

ASTM C597—Standard Test Method for Pulse Velocity Through Concrete.

ASTM C805/C805M—Standard Test Method for Rebound Number of Hardened Concrete.

ASTM C1064/C1064M—Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete.

ASTM C1077—Standard Practice for Agencies Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Testing Agency Evaluation.

ASTM D698—Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft<sup>3</sup> [600 kN-m/m<sup>3</sup>]).

ASTM D1143/D1143M—Standard Test Methods for Deep Foundations Under Static Axial Compressive Load.

ASTM D4945—Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.

ASTM E329—Standard Specification for Agencies Engaged in Construction Inspection, Testing, or Special Inspection.

AWS 2.4—Standard Symbols for Welding, Brazing, and Nondestructive Examination.

AWS\* B2.1/B2.1M-BMG—Base Metal Grouping for Welding Procedure and Performance Qualification.

AWS D1.1/D1.1M—Structural Welding Code—Steel.

AWS D1.4/D1.4M—Structural Welding Code—Reinforcing Steel.

AWS QC1—Standard for AWS Certification of Welding Inspectors.

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\* American Welding Society, 550 Northwest LeJeune Road, Miami, FL 33126.

CAN/CSA\* W178.2—Certification of Welding Inspectors.

CSA G40.20-13/G40.21—General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel.

NEMA† 250—Enclosures for Electrical Equipment (1,000 Volts Maximum).

NFPA‡ 70—National Electrical Code®.

NFPA 780—Standard for the Installation of Lightning Protection Systems.

OSHA§—Occupational Safety and Health Standards, 29 CFR, Part 1910.

PCI¶ MNL 116—Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products.

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## SECTION 3: DEFINITIONS

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The following definitions are used in this standard:

1. *Active fault*: A fault determined to be active from properly substantiated data (e.g., most recent mapping of faults by the US Geological Survey).
2. *Bottom capacity line (BCL)*: The elevation above which the tank capacity is contained, not lower than the top of the inlet/outlet pipe or silt stop, if provided.
3. *Capacity*: The net volume contained between the bottom capacity line and top capacity line.
4. *Characteristic earthquake*: An earthquake assessed for an active fault having a magnitude equal to the best estimate of the maximum magnitude capable of occurring on the fault but not less than the largest magnitude that has occurred historically on the fault.
5. *Composite elevated tank*: The entire structure: foundation, concrete support structure, steel tank, and accessories.
6. *Concrete support structure*: All concrete elements above the top of the foundation that provide primary support of the tank.
7. *Deep foundation*: Piles and pile cap that transfer load from the concrete support structure to soil or rock strata by end bearing, by skin friction or adhesion, or by both.

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\* Canadian Standards Association, 5060 Spectrum Way, Suite 100, Mississauga, ON, Canada L4W 5N6.

† National Electrical Manufacturers Association, 1300 North 17th Street, Suite 1752, Rosslyn, VA 22209.

‡ National Fire Protection Association, 1 Batterymarch Park, Quincy, MA 02169.

§ Occupational Safety and Health Administration, 200 Constitution Avenue, NW, Washington, DC 20210.

¶ Precast/Prestressed Concrete Institute, 209 West Jackson Boulevard, Suite 500, Chicago, IL 60606.

8. *Interface region*: Those portions of the support wall, tank floor, ring-beam, and steel tank affected by the transfer of forces from the tank floor and steel tank to the support wall.

9. *Intermediate floor*: A structural floor located above the slab-on-grade.

10. *Manufacturer*: The party that manufactures, fabricates, or produces materials or products.

11. *Pile*: Driven, augered, or drilled foundation elements that transfer load from the pile cap to soil or rock strata by end bearing, by skin friction or adhesion, or by both.

12. *Pile cap*: Concrete element that transfers load from the concrete support structure to the supporting piles.

13. *Purchaser*: The person, company, or organization that purchases any materials or work to be performed.

14. *Ringbeam*: Concrete element connecting the support wall, tank floor, and steel tank.

15. *Risk-targeted maximum considered earthquake (MCE<sub>R</sub>) ground motion response acceleration*: The most severe earthquake effects considered by this standard as defined in Sec. 4.2.7.5.2 and Sec. 4.2.7.6.2.

16. *Rustication*: A shallow indentation formed in the concrete surface to provide an architectural effect.

17. *Shallow foundation*: Concrete element that transfers load from the concrete support structure directly to soil or rock.

18. *Steel liner or liner*: Nonstructural welded steel plate placed over a concrete tank floor and welded to the steel tank to provide a watertight container. Considered part of the steel tank.

19. *Steel tank or tank*: Circular welded steel structure composed of a roof, shell outside the support wall, liner or structural bottom, and elements that attach the steel tank to the concrete support structure.

20. *Support wall or wall*: Concrete wall supporting the steel tank that extends from the foundation to the interface region.

21. *Tank floor*: The structural element that supports the tank contents inside the support wall.

22. *Top capacity line (TCL)*: The elevation of the lip of the overflow.

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## SECTION 4: GENERAL REQUIREMENTS FOR ANALYSIS AND DESIGN

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### Sec. 4.1 General

4.1.1 *Scope.* This section covers design loads and general requirements for analysis and design.

4.1.2 *Notation.*

4.1.2.1 *Loads.* The following symbols and notation represent applied loads in Sec. 4.3:

$D$  = dead load

$E_h$  = horizontal seismic load effect

$E_v$  = vertical seismic load effect

$F$  = water load

$G$  = eccentric load due to dead and water loads

$L$  = live load on intermediate floors

$S$  = larger of snow load or roof live load

$T$  = creep, shrinkage, differential settlement, or temperature effects

$W$  = wind load

4.1.2.2 *Variables.* The following symbols and notation represent variables in Sec. 4.1 and 4.2:

$A_f$  = vertical projected area of a portion of the structure, ft<sup>2</sup>

$C_f$  = wind force drag coefficient

$C_r$  = roof slope factor

$C_s$  = seismic response coefficient

$C_u$  = coefficient for upper limit on calculated period

$d_s$  = height of the seismic sloshing wave, ft

$D_F$  = Diameter of the free water surface, ft

$D_w$  = mean diameter of support wall, in.

$e_g$  = vertical load eccentricity, in.

$e_o$  = minimum vertical load eccentricity, in.

$E_c$  = modulus of elasticity of concrete, psi

$f_{ca}$  = average service level compression stress in the support wall due to gravity loads for load combination S1.1, psi

$F_a$  = acceleration-based site coefficient at short period

$F_i$  = portion of factored seismic force  $V$  acting at level  $i$ , lb

$F_v$  = velocity-based site coefficient at 1-second period

$F_w$  = wind force acting on a portion of the structure, lb

$F_x$  = portion of factored seismic force  $V$  acting at level  $x$ , lb

$g$  = acceleration due to gravity

$G_f$  = wind gust effect factor

$h/D$  = wind force drag coefficient height-to-diameter ratio

$H_F$  = equivalent height of a cylinder with diameter  $D_F$  containing the total stored water volume, ft

$I$  = importance factor

$K_z$  = wind velocity exposure coefficient evaluated at height  $z$

$l_g$  = distance from base to centroid of stored water, in.

$l_{ga}$  = distance from base to TCL minus one-half the operating head range; approximate distance from base to centroid of stored water, in.

$l_i$  = distance from base to level of  $F_i$ , ft

$l_w$  = distance from base to top of concrete support wall, ft

$l_x$  = distance from base to level of  $F_x$ , ft

$M_o$  = factored seismic moment at base, ft-lb

$M_x$  = factored seismic moment at level  $x$ , ft-lb

$\bar{N}$  = average field standard penetration resistance for the top 100 ft (30 m)

$\bar{N}_{ch}$  = average standard penetration resistance for cohesionless soil layers for the top 100 ft (30 m)

$p_g$  = ground snow load, psf

$p_z$  = wind pressure at height  $z$  above ground level, psf

$q_s$  = wind velocity pressure, psf

$R$  = response modification coefficient

$S_a$  = design spectral response acceleration at any period

$S_{ac}$  = spectral acceleration of the sloshing liquid (convective component) based on the sloshing period  $T_c$  and 0.5 percent damping

$S_{aM}$  = site-specific  $MCE_R$  spectral response acceleration at any period

$S_{DS}$  = design, 5 percent damped, spectral response acceleration parameter at short periods

$S_{D1}$  = design, 5 percent damped, spectral response acceleration parameter at a period of 1 second

$S_{MS}$  = the  $MCE_R$ , 5 percent damped, spectral response acceleration parameter at short periods adjusted for site class effects

$S_{M1}$  = the  $MCE_R$ , 5 percent damped, spectral response acceleration parameter at a period of 1 second adjusted for site class effects



- $S_S$  = mapped  $MCE_R$ , 5 percent damped, spectral response acceleration parameter at short periods as defined in Sec. 4.2.7.5.1  
 $S_1$  = mapped  $MCE_R$ , 5 percent damped, spectral response acceleration parameter at a period of 1 second as defined in Sec. 4.2.7.5.1  
 $\bar{s}_u$  = average undrained shear strength in top 100 ft (30 m), psf  
 $T$  = fundamental period of vibration of the structure, sec  
 $T_a$  = approximate fundamental period of vibration of the structure, sec  
 $T_c$  = natural period of the first (convective) mode of sloshing, sec  
 $T_L$  = long-period transition period determined from chapter 22 of ASCE 7, sec  
 $\bar{v}_s$  = average shear wave velocity at small shear strains in top 100 ft (30 m), ft/sec  
 $V$  = total design seismic lateral force or shear at the base, lb  
 $V_b$  = basic wind speed, mph  
 $w_f$  = uniformly distributed flat roof snow load, psf  
 $w_i$  = portion of total mass the centroid of which is at level  $i$ , a distance  $l_i$  above the base, lb  
 $w_s$  = uniformly distributed sloped roof snow load, psf  
 $w_x$  = portion of effective seismic weight  $W$  the centroid of which is at level  $x$ , a distance  $l_x$  above the base, lb  
 $W$  = effective seismic weight, lb  
 $z$  = height above ground level, ft  
 $z_g$  = nominal height of the atmospheric boundary layer, ft  
 $\alpha$  = 3-second gust speed power law exponent  
 $\theta_g$  = estimated foundation tilt, degrees  
 $\theta_r$  = roof slope at a point measured from the horizontal, degrees

## Sec. 4.2 Design Loads

4.2.1 *Minimum loads.* The structure shall be designed for the loads defined in this standard, unless larger loads are specified.

4.2.2 *Dead load.* Dead load shall be the weight of all permanent structural components, attachments, and equipment. Unit weight of material shall be based on test data of materials, when available. Otherwise, unit weights shall be 490 lb/ft<sup>3</sup> (7,850 kg/m<sup>3</sup>) for steel, 150 lb/ft<sup>3</sup> (2,400 kg/m<sup>3</sup>) for reinforced concrete, and 100 lb/ft<sup>3</sup> (1,600 kg/m<sup>3</sup>) for soil.

4.2.3 *Water load.* Water load shall be the weight of the water when the tank is filled to the TCL. The unit weight for water shall be 62.4 lb/ft<sup>3</sup> (1,000 kg/m<sup>3</sup>).

4.2.4 *Live load.* Distributed and concentrated live loads acting on the tank roof, access areas, elevated platforms, and intermediate floors shall be in accordance with ASCE 7 unless otherwise specified.

4.2.5 *Snow load.*

4.2.5.1 *Scope.* This section defines minimum service snow loads for design. It is applicable to conical roofs and curved roofs that concave downward without steps or abrupt changes in elevation.

4.2.5.2 *Ground snow load.* Ground snow load  $p_g$  shall be based on an extreme value statistical analysis of weather records using a 2 percent annual probability of exceedance (a 50-year mean recurrence interval). In the contiguous United States and Alaska the ground snow load  $p_g$  shall be determined from ASCE 7 Figure 7-1, "Ground Snow Loads,  $p_g$ , for the United States (lb/ft<sup>2</sup>)."

4.2.5.3 *Flat roof snow load.* Unless otherwise specified, the uniformly distributed snow load  $w_f$  acting on a flat roof shall be

$$w_f = 0.9 p_g \geq 25 \text{ psf (1.2 kPa)} \quad \text{for } p_g > 20 \text{ psf (960 Pa)} \quad (\text{Eq 4-1a})$$

$$w_f = 1.2 p_g \geq 15 \text{ psf (720 Pa)} \quad \text{for } p_g \leq 20 \text{ psf (960 Pa)} \quad (\text{Eq 4-1b})$$

4.2.5.4 *Sloped roof snow load.* Portions of the tank having a slope  $\theta_r$  greater than 70 degrees shall be considered free of snow load. The sloped roof snow load  $w_s$  at any point on the roof shall be

$$w_s = C_r w_f \quad (\text{Eq 4-2})$$

where:

$$C_r = 1.75 - \theta_r / 40 \quad 0 < C_r \leq 1 \quad (\text{Eq 4-3})$$

For curved roofs, the distribution of snow load may be assumed to vary linearly between roof slopes of 30 degrees and 70 degrees.

4.2.6 *Wind load.*

4.2.6.1 *Scope.* This section defines the minimum factored wind loads for design of the main wind-force-resisting system. Larger wind loads shall be used when specified.

4.2.6.2 *Basic wind speed.* Basic wind speed  $V_b$  shall be based on a 3-second gust speed at 33 ft (10 m) above ground corresponding to approximately a 3 percent probability of exceedance in 50 years (mean recurrence interval = 1,700 years). In the contiguous United States and Alaska, the basic wind speed  $V_b$  shall be determined from ASCE 7-10, Figure 26.5-1B, "Basic Wind Speeds for Risk Category III and IV Buildings and Other Structures," and shall not be less than 120 mph (54 m/sec).

In special wind regions and areas outside the contiguous United States and Alaska, determine basic wind speed in accordance with ASCE 7-10, Sec. 26.5-2 and 26.5-3.

4.2.6.3 Design wind force. Total factored wind force acting on the main wind-force-resisting system is the sum of the forces  $F_w$  acting on each portion of the structure determined by Eq 4-4.

The wind force  $F_w$  acting on any portion of the structure is

$$F_w = C_f p_z A_f \quad \text{not less than 48 psf (2,300 Pa) times } C_f A_f \quad (\text{Eq 4-4})$$

NOTE: Design wind pressure  $C_f p_z$  used in Eq 4-4 corresponding to a basic wind speed  $V_b$  of 120 mph (54 m/sec) may be determined from Tables A.1a and A.1b in appendix A.

The wind force drag coefficient  $C_f$  is

$$C_f = 0.5 + \frac{h/D - 1}{60} \quad 0.5 \leq C_f \leq 0.6 \quad (\text{Eq 4-5})$$

The ratio of structure height-to-diameter ratio  $h/D$  in Eq 4-5 shall be calculated using:

- Height  $h$  measured from grade to a point on the roof, where roof slope  $\theta_r$  is 30 degrees or less
- Diameter  $D$  equal to maximum tank diameter

Wind pressure  $p_z$  at height  $z$  above ground level is

$$p_z = q_s G_f K_z \quad (\text{Eq 4-6})$$

Wind velocity pressure  $q_s$  is

$$q_s = 0.00256 V_b^2 \quad (\text{Eq 4-7})$$

Wind gust effect factor  $G_f$  is 0.85 for structure period 1.0 second or less, and  $G_f$  is 1.0 when structure period exceeds 1.0 second.

Velocity exposure height coefficient  $K_z$  shall be determined by Eq 4-8a/4-8b or from ASCE 7 Table 6-3. Unless otherwise specified, Exposure Category C shall be used.

$$K_z = 2.01 (z/z_g)^{2/\alpha} \quad \text{for } z_{15} \leq z \leq z_g \quad (\text{Eq 4-8a})$$

$$K_z = 2.01 (z_{15}/z_g)^{2/\alpha} \quad \text{for } z < z_{15} \quad (\text{Eq 4-8b})$$

where  $z_{15} = 15$  ft (4.6 m), and values of  $\alpha$  and  $z_g$  are from Table 1.

4.2.6.4 Other wind loads. Wind forces acting on specified accessories or other components attached to the composite elevated tank shall be considered.

1. Minimum wind forces acting on projecting accessories or components shall comply with ASCE 7. These loads may be neglected in design of the main

**Table 1** Coefficients for calculation of velocity exposure height coefficient  $K_z$ 

Exposure Category	$\alpha$	$z_g$	
		ft	(m)
C	9.5	900	(274.32)
D	11.5	700	(213.36)

wind-force-resisting system when they increase overturning moment by 5 percent or less.

2. Minimum wind forces on shrouds that totally enclose the structure for environmental protection shall be based on basic wind speed  $V_b$  of 67 mph (30 m/s), acting on projected area of tank height by a width equal to maximum tank diameter plus 6 ft (1.8 m).

Supporting elements and connections for projecting accessories or components shall have sufficient strength to resist applied wind loads.

#### 4.2.7 Seismic design criteria.

4.2.7.1 Scope. This section defines criteria to be used in design of the main lateral-force-resisting system. Seismic analysis is not required where mapped maximum spectral response acceleration at 1-second period  $S_1$  is less than or equal to  $0.04g$  and mapped maximum spectral response acceleration at short periods  $S_S$  is less than or equal to  $0.15g$ .

4.2.7.2 Site classification. The site class shall be determined in accordance with the classification procedure given in chapter 20 of ASCE 7.

4.2.7.3 Site coefficients. Site coefficients  $F_a$  and  $F_v$  shall be determined from Tables 2a and 2b, respectively. For intermediate values of  $S_S$  and  $S_1$ , the higher value or straight-line interpolation may be used to determine coefficients  $F_a$  and  $F_v$ .

4.2.7.4 Ground motions. Ground motions shall be determined in accordance with Sec. 4.2.7.5 or Sec. 4.2.7.6. For structures on Site Class F sites, ground motions shall be determined by site-specific procedures in accordance with Sec. 4.2.7.6.

4.2.7.5 General procedure. Risk-targeted maximum considered earthquake ( $MCE_R$ ) and design spectral response.

**Table 2a Site coefficient  $F_a$** 

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Site-specific response analysis required; see Sec. 4.2.7.6.				

**Table 2b Site coefficient  $F_v$** 

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-Second Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Site-specific response analysis required; see Sec. 4.2.7.6.				

4.2.7.5.1 Mapped acceleration parameters.\* Ground motion acceleration shall be based on a 2 percent probability of exceedance in 50 years with 5 percent critical damping. The mapped risk-adjusted maximum considered earthquake spectral response acceleration parameters  $S_S$  and  $S_1$  shall be determined from the USGS Earthquake Hazards Program—National Seismic Hazard Mapping Project, 2008 database for Site Class B. Alternatively,  $S_S$  and  $S_1$  may be determined by interpolation from  $MCE_R$  maps in chapter 22 of ASCE 7.

4.2.7.5.2 Maximum considered earthquake. Maximum considered earthquake spectral response acceleration at short periods  $S_{MS}$  and at 1-second period  $S_{M1}$  adjusted for site class effects are

\* Values of mapped acceleration parameters and seismic design parameters are provided at the USGS Web site at [earthquake.usgs.gov/designmaps](http://earthquake.usgs.gov/designmaps), or through the SEI Web site at [content.seinstitute.org](http://content.seinstitute.org).

$$S_{MS} = F_a S_S \quad (\text{Eq 4-9a})$$

$$S_{M1} = F_v S_1 \quad (\text{Eq 4-9b})$$

4.2.7.5.3 Design spectral response acceleration. Design spectral response acceleration at short periods  $S_{DS}$  and at 1-second period  $S_{D1}$  are

$$S_{DS} = \frac{2}{3} S_{MS} \quad (\text{Eq 4-10a})$$

$$S_{D1} = \frac{2}{3} S_{M1} \quad (\text{Eq 4-10b})$$

Where a design response spectrum is required and site-specific procedures are not used, the design spectral acceleration at any period is

$$S_a = S_{D1}/T \leq S_{DS} \quad (\text{Eq 4-11})$$

4.2.7.6 Site-specific procedure. Risk-targeted maximum considered earthquake ( $MCE_R$ ) and design spectral response.

4.2.7.6.1 Ground motion hazard analysis. When site-specific ground motions are used or required, a ground motion hazard analysis for the site shall be performed in accordance with chapter 21 of ASCE 7 and documented in a report. Site-specific  $MCE_R$  spectral response acceleration at any period shall be determined using probabilistic procedures or deterministic procedures, or both. Probabilistic  $MCE_R$  spectral response accelerations at each period shall be based on a 2 percent probability of exceedance in 50 years with 5 percent critical damping. Deterministic  $MCE_R$  response accelerations at each period shall be taken as 150 percent of the largest median 5 percent damped spectral acceleration computed at that period for characteristic earthquakes on all known active faults within the region.

4.2.7.6.2 Risk-targeted maximum considered earthquake ( $MCE_R$ ). Site-specific  $MCE_R$  spectral response acceleration at any period  $S_{aM}$  shall be taken as the lesser of the spectral response accelerations from the probabilistic  $MCE_R$  and the deterministic  $MCE_R$ . The deterministic  $MCE_R$  ground motion response spectrum ordinates shall not be less than the value determined by Eq 4-12.

$$S_{aM} = 0.6 F_v / T \leq 1.5 F_a \quad (\text{Eq 4-12})$$

For Site Class F sites, values of  $F_a$  and  $F_v$  used in Eq 4-12 shall be for Site Class E from Tables 2a and 2b, respectively.

4.2.7.6.3 Site-specific design spectral response acceleration. Site-specific design spectral response acceleration  $S_a$  is the larger of 80 percent of the value determined by Eq 4-11 and the value determined from Eq 4-13.

$$S_a = (\frac{2}{3}) S_{aM} \quad (\text{Eq 4-13})$$

Site-specific  $MCE_R$  spectral response acceleration at any period  $S_{aM}$  is in accordance with Sec. 4.2.7.6.2.

4.2.7.6.4 Site-specific acceleration parameters. Site-specific design spectral response acceleration parameters at short periods and at 1 second are

- $S_{DS}$  is the spectral acceleration  $S_a$  obtained from Sec. 4.2.7.6.3 at a period of 0.2 seconds, but not less than the larger of 90 percent of the peak spectral acceleration  $S_a$  at any period greater than 0.2 seconds, and 80 percent of the value obtained from Eq 4.10a.
- $S_{D1}$  is the larger value of spectral acceleration  $S_a$  obtained from Sec. 4.2.7.6.3 at 1 second and two times the spectral acceleration  $S_a$  at 2 seconds, but not less than 80 percent of the value obtained from Eq 4.10b.

4.2.7.7 Importance factor. An importance factor  $I$  of 1.5 shall be used for seismic loads. The importance factor  $I$  may be reduced to 1.25 when the structure is not an essential facility used for fire protection or potable water supply.

4.2.7.8 Seismic design category (SDC). Determine seismic design category in accordance with Sec. 11.6 of ASCE 7 when required for design or detailing of elements not expressly covered in the standard. Sites where  $S_S$  is 0.15g or less and  $S_1$  is 0.04g or less shall be assigned to SDC A.

#### 4.2.8 Seismic load.

4.2.8.1 Scope. This section defines the minimum factored seismic loads for design of the main lateral-force-resisting system.

##### 4.2.8.2 Seismic load effects.

4.2.8.2.1 Vertical. Factored vertical seismic load effect  $E_v$  due to vertical acceleration at the base shall not be less than

$$E_v = 0.2 S_{DS} W \quad (\text{Eq 4-14})$$

The seismic coefficient  $S_{DS}$  is in accordance with Sec. 4.2.7.5.3 or Sec. 4.2.7.6.4, and the effective seismic weight  $W$  is in accordance with Sec. 4.2.8.3.1.

Vertical distribution of  $E_v$  is proportional to the vertical distribution of the structure weight.

4.2.8.2.2 Horizontal. Factored horizontal seismic load effect  $E_h$ , the shear and the resulting moment acting on any horizontal section of the structure, shall be determined in accordance with Sec. 4.2.8.4 and Sec. 4.2.8.5.

4.2.8.2.3 *P*-delta effects. *P*-delta effects may be ignored when the increase in seismic overturning moment is less than 10 percent of the moment without *P*-delta effects.

4.2.8.3 Seismic base shear. Factored seismic base shear  $V$  acting in any direction shall be in accordance with Sec. 4.2.8.3.1 and Sec. 4.2.8.3.2.

4.2.8.3.1 Equivalent lateral force procedure. The equivalent lateral force procedure may be used for all structures and shall be based on a fixed-base, single degree-of-freedom model. All mass, including the water, shall be considered rigid.

Factored seismic base shear  $V$  is

$$V = C_s W \quad (\text{Eq 4-15})$$

Seismic response coefficient  $C_s$  is the larger value determined from Equations 4-16a through 4-16c.

$$C_s = \frac{S_a I}{R} \quad (\text{Eq 4-16a})$$

The value of  $S_a$  used in Eq 4-16a is for design spectral response acceleration determined in Sec. 4.2.7.5.3 or Sec. 4.2.7.6.3 at fundamental structure period  $T$  from Sec. 4.2.8.7.

The seismic response coefficient  $C_s$  shall not be less than

$$C_s = 0.044 S_{DS} I \geq 0.01 \quad (\text{Eq 4-16b})$$

The value of  $S_{DS}$  in Eq 4-16b is from Sec. 4.2.7.5.3 or Sec. 4.2.7.6.4.

When  $S_1$  determined in Sec. 4.2.7.5.1 is equal to or greater than 0.6g, seismic response coefficient  $C_s$  shall not be less than

$$C_s = \frac{0.5 S_1 I}{R} \quad (\text{Eq 4-16c})$$

Effective seismic weight  $W$  shall include dead load above the base; water load; and 25 percent of live load from intermediate floors.

Importance factor  $I$  is determined in Sec. 4.2.7.7.

4.2.8.3.2 Alternative procedures. Analysis complying with Sec. 15.7.10 of ASCE 7 may be used in lieu of the equivalent lateral force procedure of Sec. 4.2.8.3.1 to determine seismic base shear  $V$ .

When soil-structure interaction is used, the effective damping factor for the structure-foundation system shall be computed using the procedure provided in ASCE 7, chapter 19, but shall not exceed 15 percent.



Fluid-structure interaction may be used when the following requirements are met:

1. The sloshing period is greater than three times the natural period of the tank with confined liquid (rigid mass) and supporting structure.
2. The sloshing mechanism (percentage of convective mass and centroid) is determined for the specific configuration of the tank by detailed fluid-structure interaction analysis or testing.
3. The response modification factor for the convective component is 1.5 or less, and the spectral acceleration of the convective component is based on 0.5 percent damping.

Factored base shear  $V$  determined by alternative procedures shall not be less than  $0.75C_sW$ . Seismic response coefficient  $C_s$  and effective seismic weight  $W$  shall be determined by the equivalent lateral force procedure of Sec. 4.2.8.3.1.

4.2.8.4 Seismic force distribution. Factored seismic base shear  $V$  shall be distributed over the height of the structure in proportion to the structure weight by

$$F_x = V \frac{w_x}{\sum_{i=1}^n w_i} \quad (\text{Eq 4-17})$$

4.2.8.5 Seismic moment.

4.2.8.5.1 Overturning moment at base. Factored base overturning moment  $M_o$  is

$$M_o = \sum_{i=1}^n (F_i l_x) \quad (\text{Eq 4-18})$$

4.2.8.5.2 Seismic moment at level  $x$ . Factored seismic moment  $M_x$  acting at any level of the structure is the larger of

$$M_x = \sum_{i=1}^x F_i (l_i - l_x) \quad (\text{Eq 4-19a})$$

or

$$M_x = M_o \left( 1 - 0.75 \frac{l_x}{l_w} \right) \quad (\text{Eq 4-19b})$$

4.2.8.6 Response modification coefficient. The response modification coefficient  $R$  used in design shall not exceed 3.0.

4.2.8.7 Structure period.

4.2.8.7.1 Fundamental period. The fundamental period of vibration  $T$  of the structure shall be established using the elastic structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis.

The period  $T$  used to calculate the seismic response coefficient  $C_r$  in Sec. 4.2.8.3.1 shall be determined from the lesser of

$$T \leq C_u T_a \quad (\text{Eq 4-20a})$$

or

$$T \leq 2.5 \text{ seconds} \quad (\text{Eq 4-20b})$$

4.2.8.7.2 Approximate fundamental period. The approximate fundamental period  $T_a$  shall be determined from one of the following equations:

$$T_a = \frac{10.26 l_g}{D_w} \sqrt{\frac{f_{ca} l_g}{E_c g}} \quad (\text{Eq 4-21a})$$

$$T_a = \frac{0.16 l_{ga}}{D_w} \sqrt{\frac{l_g}{g}} \quad (\text{Eq 4-21b})$$

Terms  $D_w$ ,  $E_c$ ,  $f_{ca}$ ,  $g$ ,  $l_g$ , and  $l_{ga}$  are in consistent units.

4.2.8.7.3 Coefficient for upper limit on calculated period. The coefficient for upper limit on calculated period  $C_u$  shall be determined from Table 3.

4.2.8.8 Sloshing. Sloshing of stored water due to seismic acceleration shall be considered by

- a. Providing minimum freeboard in accordance with Sec. 4.2.8.8.1; or
- b. Designing the roof and supporting structure to contain the sloshing water.

4.2.8.8.1 Minimum freeboard. Minimum freeboard shall comply with the following where seismic design is required:

- a. Not less than the sloshing wave height  $d_s$  for Risk Category IV structures.
- b. Not less than  $0.7d_s$  for Risk Category III structures when  $S_{DS} > 0.33g$ .
- c. No minimum freeboard is required for Risk Category III structures when

$S_{DS} \leq 0.33g$ .

Height of the sloshing wave  $d_s$  shall be computed by

**Table 3 Coefficient for upper limit on calculated period  $C_u$ \***

Design Spectral Response Acceleration at 1 second, $S_{D1}$	Coefficient $C_u$
$\geq 0.4$	1.4
0.3	1.4
0.2	1.5
0.15	1.6
$\leq 0.1$	1.7

\* Interpolation may be used for intermediate values.

$$d_s = 0.5 D_F I S_{ac} \quad (\text{Eq 4-22})$$

Spectral acceleration of the sloshing liquid  $S_{ac}$  is the smaller value from Eq 4-23a and Eq 4-23b. Equation 4-23b is applicable only when the long-period transition period  $T_L$  is determined and  $T_c > T_L$ .

$$S_{ac} = 1.5 S_{D1}/T_c \leq 1.5 S_{DS} \quad (\text{Eq 4-23a})$$

$$S_{ac} = 1.5 S_{D1} T_L/(T_c)^2 \quad (\text{Eq 4-23b})$$

Natural period of the first (convective) mode of sloshing  $T_c$  is computed by

$$T_c = 2\pi \sqrt{\frac{D_F}{3.68 \tanh(3.68 H_F/D_F)}} \quad (\text{Eq 4-24})$$

#### 4.2.9 Other loads.

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

$$e_g \geq e_o + \frac{l_g}{400} \quad (\text{Eq 4-25a})$$

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

$$e_g \geq e_o + l_g \left( \frac{1}{800} + \tan \theta_g \right) \quad (\text{Eq 4-25b})$$

4.2.9.2 Construction loads. Temporary loads resulting from construction activity shall be considered in the design of structural components.

4.2.9.3 Future construction. When future construction such as the addition of intermediate floors is specified, the load effects shall be included in the original design. Future construction dead and live loads shall be included in the Group 1 load combinations (see Sec. 4.3.3.1 and Sec. 4.3.4.1). Only that portion of the dead load existing at the time of original construction shall be included in the Group 2 load combinations (see Sec. 4.3.3.2 and Sec. 4.3.4.2).

### Sec. 4.3 Analysis and Design

4.3.1 Analysis. The design of the composite elevated tank and its components shall be based on the results of a detailed structural analysis, which shall provide the applicable axial loads, shears, and moments to be used in design. Methods of analysis may be based on classical theory, simplified mathematical models, or

numerical solutions using finite element, finite differences, or numerical integration techniques. The analysis shall consider the effects of restraint at the boundaries of shell elements.

4.3.2 *Design methods.* Design the steel tank by the working stress method using the unfactored load combinations in Sec. 4.3.3. Design the concrete support structure and concrete foundation by the strength design method to resist the factored load combinations in Sec. 4.3.4. Size the foundations using the unfactored load combinations in Sec. 4.3.3 in accordance with Sec. 7.3.1.

4.3.3 *Unfactored load combinations.* Load combinations for the working stress method shall comply with the following:

4.3.3.1 Group 1 load combinations. The allowable load shall not be less than

Load combination:	S1.1	$D + F$
	S1.2	$D + F + G + S + L$
	S1.3	$D + F + G + L + 0.6W$
	S1.4	$D + F + G + L + 0.7E_h + 0.7E_v$

4.3.3.2 Group 2 load combinations. Where  $D$ ,  $L$ , or  $F$  reduce the effect of  $W$ ,  $E_h$ , or  $E_v$ , as in uplift produced by overturning moment, the allowable load shall not be less than

Load combination:	S2.1	$D + 0.6W$
	S2.2	$D + F + 0.7E_h + 0.7E_v$

4.3.3.3 Differential settlement, creep, shrinkage, and temperature. Where structural effects of differential settlement, creep, shrinkage, or temperature effects are significant,  $1.0T$  shall be included with load combinations S1.1 and S1.2, and  $0.75T$  shall be included with load combinations S1.3 and S1.4. Where structural effects  $T$  are significant,  $0.75T$  shall be included with Group 2 load combinations when  $T$  is additive to  $W$ ,  $E_h$ , or  $E_v$ .

4.3.4 *Factored load combinations.* Load factors and load combinations for the strength design method shall comply with the following:

4.3.4.1 Group 1 load combinations. Where the structural effects of applied loads are cumulative, the required strength shall not be less than

Load combination:	U1.1	$1.4(D + F)$
	U1.2	$1.2(D + G) + 1.5F + 1.6(S + L)$
	U1.3	$1.2(D + F) + 0.5(S + G) + L + W$
	U1.4	$1.2(D + F) + 0.5G + L + E_h + E_v$

4.3.4.2 Group 2 load combinations. Where  $D$ ,  $L$ , or  $F$  reduce the effect of  $W$ ,  $E_h$ , or  $E_v$ , as in uplift produced by overturning moment, the required strength shall not be less than

Load combination:	U2.1	$0.9 D + W$
	U2.2	$0.9 D + F + E_h + E_v$

4.3.4.3 Differential settlement, creep, shrinkage, and temperature. Where structural effects of differential settlement, creep, shrinkage, or temperature effects are significant,  $1.2T$  shall be included with load combinations U1.2 and U1.3, and  $1.0T$  shall be included with load combination U1.4. Where structural effects  $T$  are significant,  $1.0T$  shall be included with Group 2 load combinations when  $T$  is additive to  $W$ ,  $E_h$ , or  $E_v$ .

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## SECTION 5: STEEL TANK

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### Sec. 5.1 General

5.1.1 *Scope.* This section covers materials, design, and construction of the steel tank.

5.1.2 *Notation.* The following symbols are used to represent variables in this section:

$A_1$  = loaded area, in.<sup>2</sup>

$A_2$  = area of the lower base of the largest frustrum contained within the supporting concrete, having side slopes of 1 vertical to 2 horizontal and loaded area  $A_1$  for its upper base, in.<sup>2</sup>

$C_p$  = elastic buckling coefficient for pressure stabilization

$C_o$  = elastic buckling coefficient

$D_s$  = diameter of cylindrical shell, in.

$f'_c$  = specified compressive strength of concrete, psi

$e_x$  = local imperfection tolerance, in.

$E_s$  = modulus of elasticity of steel, psi

$f_h$  = circumferential membrane tension stress due to hydrostatic pressure, psi

$F_{cr}$  = critical buckling strength, psi

$F_L$  = meridional compression strength, psi

$F_{xe}$  = maximum meridional compression strength, psi

$F_y$  = minimum yield point or minimum yield strength specified in referenced material standards, psi

$F_p$  = allowable bearing stress on concrete, psi

$K_\phi$  = meridional compression strength factor

$L$  = centerline spacing of rafters at maximum radius, in.

$L_x$  = gauge length for measuring local imperfections, in.

$p$  = hydrostatic pressure at point under consideration, psi

$R$  = radius of the shell, normal to the plate at the point under consideration and measured from the centerline of the plate to the axis of revolution, in.

$(R/t)_c$  = radius-to-thickness ratio at which buckling changes from elastic to inelastic

$t$  = plate thickness less the specified corrosion allowance, in.

$t_{base}$  = plate thickness determined by Eq 5-3a and 5-3b with  $F_y$  equal to 36,000 psi (248 MPa), in.

$w_r$  = roof design load, psf

$\eta$  = plasticity reduction factor

5.1.3 *Configuration.* Provide an all-welded steel tank comprising one or more axisymmetric shell courses and a roof. Provide welded steel plate liners over concrete floors.

5.1.4 *Modulus of elasticity.* Modulus of elasticity  $E_s$  for steel used in design shall be 29,000,000 psi (200 GPa).

## Sec. 5.2 Material Requirements

5.2.1 *General.* All materials incorporated into the steel tank shall be previously unused. Unless otherwise specified, materials shall comply with the requirements of this standard.

5.2.2 *Steel plate.* Steel plate shall conform to the requirements of Table 4. Thickness limits apply to shell elements where primary membrane tension or compression stress exceeds 6,000 psi (41 MPa).

5.2.3 *Other structural steel.* Other steel materials and products shall comply with requirements of ANSI/AISC 360.

5.2.4 *Filler metals and welding consumables.* Filler metals and welding consumables shall be as required by weld procedure specifications in Sec. 5.4.

## Sec. 5.3 Analysis and Design

5.3.1 *Analysis.* Perform analysis to determine the following:

1. In-plane membrane forces in shell elements and boundary elements such as tension and compression rings, where shear and flexure are not significant and membrane theory of shells is applicable.
2. Axial load, shear, and flexure in other elements.

**Table 4 Steel plate requirements**

Specification	Grade	Minimum Yield Strength, $F_y$		Maximum Thickness, Tension		Maximum Thickness, Compression	
		(psi)	(MPa)	(in.)	(mm)	(in.)	(mm)
ASTM A36	—	36,000	248	2*	51*	4*,†,‡,§	100*,†,‡,§
ASTM A516	60	32,000	221	1½	38	4†	100†
	65	35,000	241	1½	38	4†	100†
	70	38,000	262	1½	38	4†	100†
ASTM A537	CL 1	50,000	345	1½	38	2½†	64†
ASTM A572	42	42,000	290	1½*	38*	4*,†	100*,†
	50	50,000	345	1½*	38*	4*,†	100*,†
ASTM A573	58	32,000	221	1½	38	1½	38
	65	35,000	241	1½	38	1½	38
	70	42,000	290	1½	38	1½	38
CSA G40.21	38W	38,000	262	2	51	4†	100†
	44W	44,000	303	2	51	4†	100†
	50W	50,000	345	2	51	4†	100†

\* Comply with supplementary requirement S91 (silicon killed and fine grain practice) for thicknesses greater than 1½ in. (38 mm).

† Comply with supplementary requirement S8 (ultrasonic examination in accordance with ASTM A435) for thicknesses greater than 2 in. (51 mm).

‡ Comply with supplementary requirement S97 (steel shall be other than rimmed or capped).

§ Normalize for thicknesses greater than 2 in. (51 mm).

### 5.3.2 General design requirements.

5.3.2.1 Design method. Design the steel tank by the working stress method using the unfactored load combinations in Sec. 4.3.3, except as permitted in Sec. 5.3.4.4. Allowable stresses may be increased by one-third for load combinations in Sec. 4.3.3 that include wind or seismic loads.

5.3.2.2 Minimum thickness. Unless otherwise specified, minimum thickness of steel tank components shall be:

1. Parts not in contact with water: ⅜ in. (4.76 mm).
2. Parts in contact with water: ¼ in. (6.35 mm).
3. Shell plate thickness controlled by Sec. 5.3.5: ⅜ in. (9.52 mm).
4. Cylindrical shells:  $D_r/6,000$ , not less than ¼ in. (6.35 mm).

For structural members, thickness is defined as the mean thickness of flanges.

5.3.2.3 Corrosion allowance. Unless otherwise specified, corrosion allowance is not required. Corrosion allowance shall be added to the calculated thickness of specified elements.

### 5.3.3 Design of welds.

5.3.3.1 General. Calculated stress in welds shall not exceed allowable stresses in Sec. 5.3.3.2 times joint efficiency factor in Sec. 5.3.3.3.

5.3.3.2 Allowable stress. Allowable stresses in welds are

1. Tension or compression on effective weld area of groove welds: same as the base metal in Table 5.

2. Shear on effective area of groove and fillet welds: 0.3 times nominal tensile strength of weld metal for filler metals matching base metal strength. Stress on fillet welds shall be considered as shear regardless of direction of load application.

5.3.3.3 Joint efficiency factors. Joint efficiency factors are

1. Tension normal to effective area of groove welds: 0.80.

2. Shear on effective area of groove and fillet welds: 0.75.

3. Compression normal to effective area of groove welds, and tension or compression parallel to effective area of groove welds: 1.0.

5.3.3.4 Effective area of welds. Effective area of a weld is the weld length times effective weld size. Effective weld size shall be

1. Complete joint penetration groove welds: thickness of the thinner part joined.

2. Partial joint penetration groove welds with bevel or V-joints: depth of chamfer minus  $\frac{1}{8}$  in. (3.18 mm).

**Table 5 Allowable stresses in shell plates and structural components**

Item	Allowable Stress*	
	(psi)	(MPa)
Shell plates:		
Membrane tension	$0.5F_y \leq 18,000$	$0.5F_y \leq 124$
Membrane compression	$0.5F_L^\dagger \leq 18,000$	$0.5F_L^\dagger \leq 124$
In-plane shear	$0.4F_y \leq 14,400$	$0.4F_y \leq 99.3$
Tension and compression rings	$0.5F_y \leq 15,000$	$0.5F_y \leq 103$
Structural members and connections	AISC	AISC
Bending in base plates and roof plates	$0.67F_y \leq 24,000$	$0.67F_y \leq 165$

\* See applicable joint efficiency in Sec. 5.3.3.3.

†  $F_L$  is determined in Sec. 5.3.5.1.



3. Fillet welds: throat of weld defined as the shortest distance from the joint root to face of weld.

5.3.3.5 Weld joint details. Weld joint details shall comply with the following:

1. Shell plates in contact with water shall be butt welded with continuous complete joint penetration welds.

2. Unless otherwise specified, all exposed joints on the tank interior and exterior shall be continuously welded with structural welds or seal welds. Seal welds shall be continuous and of sufficient size to prevent cracking.

3. Maximum thickness of plate connected by lap welds is  $\frac{3}{8}$  in. (9.52 mm). Plate laps shall be minimum 1 in. (25 mm) and maximum 4 in. (100 mm) for single-welded lap joints. Minimum plate lap for double-welded lap joints subjected to calculated stress is five times the thickness of the thinner part joined.

4. Minimum length of fillet weld is the greater of four times the nominal weld size or  $1\frac{1}{2}$  in. (38 mm). Maximum size of fillet weld along edge of a connected part is the edge thickness for material less than  $\frac{1}{4}$ -in. (6.35-mm) thick, and the edge thickness minus  $\frac{1}{16}$  in. (1.59 mm) for material  $\frac{1}{4}$ -in. (6.35-mm) thick or greater.

5. Minimum size of structural fillet welds and partial penetration groove welds shall comply with Table 6. Minimum size of seal welds is  $\frac{1}{8}$  in. (3.18 mm).

6. Welding to steel members embedded in concrete shall not detrimentally affect the connected parts.

#### 5.3.4 Design of shell plates and structural components.

5.3.4.1 General. Calculated stress in shell plate and structural components shall not exceed allowable stresses in Table 5.

**Table 6 Minimum size of fillet and partial joint penetration welds**

Base Metal Thickness*		Minimum Weld Size†	
(in.)	(mm)	(in.)	(mm)
To $\frac{1}{2}$ inclusive	To 12.7 inclusive	$\frac{3}{16}$	4.76
Over $\frac{1}{2}$ to $\frac{3}{4}$	Over 12.7 to 19	$\frac{1}{4}$	6.35
Over $\frac{3}{4}$ to $1\frac{1}{2}$	Over 19 to 38	$\frac{5}{16}$	7.94
Over $1\frac{1}{2}$ to $2\frac{1}{4}$	Over 38 to 57	$\frac{3}{8}$	9.52
Over $2\frac{1}{4}$ to 6	Over 57 to 150	$\frac{1}{2}$	12.7

\* Thickness of thicker part joined.

† Fillet weld size need not exceed thickness of thinner part joined.

5.3.4.2 Shell plates. Proportion shell plates for allowable stresses in Table 5 and weld requirements in Sec. 5.3.3. Cylindrical shells shall be designed to resist external wind pressure when empty with a factor of safety of 2.0.

Openings in plate elements subject to primary membrane tension or compression stress shall be reinforced in accordance with the requirements of API 650.

5.3.4.3 Tension and compression rings. Provide a tension or compression ring at junctures in the shell where the meridional forces are discontinuous when membrane analysis is used for design. Shell plates within a distance of  $0.78\sqrt{Rt}$  from the discontinuity juncture may be assumed to participate in the ring.

5.3.4.4 Access tube. Proportion the access tube for the allowable stresses in Table 5. Access tube shell plate shall be designed to resist external water pressure with a factor of safety of 2.0.

5.3.4.5 Tank roof. Provide a plate steel roof with rafters or stiffeners, if required, to support distributed load  $w_r$  comprising the roof dead load defined in Sec. 4.2.2 plus the greater of roof live load in Sec. 4.2.4 or snow load in Sec. 4.2.5. Design the roof to prevent ponding under design load  $w_r$ . Locate all parts of the roof including rafters or stiffeners and connections above the TCL. Maximum rafter spacing  $L$  for supported roofs is determined by Eq 5-1.

$$L = \frac{2,575t}{\sqrt{w_r}} \leq 84 \text{ in.} \quad (\text{Eq 5-1})$$

Cone roofs incorporating rafters shall be designed in accordance with ANSI/AISC 360. Permissible deflection of rafters under the greater of roof live load in Sec. 4.2.4 or snow load in Sec. 4.2.5 is the rafter span divided by 240.

5.3.4.6 Steel tank connection. Design of the interface region shall comply with Sec. 6.3.6. The steel tank and concrete support structure shall be connected to provide structural continuity for applied loads. The connection shall be designed for the seismic force determined in Sec. 4.2.7.4 using  $R/I$  equal to 1.0.

Base plates bearing on concrete shall be proportioned using Eq 5-2. Supporting concrete area  $A_2$  shall not be less than two times the loaded area  $A_1$ . Allowable bearing stress on concrete  $F_p$  is

$$F_p = 0.35 f_c' \sqrt{A_2/A_1} \leq 0.70 f_c' \quad (\text{Eq 5-2})$$

5.3.4.7 Liner plate. Provide adequate support and connection details for welded liner plate over concrete floors to prevent damage to coatings and welds caused by excessive flexing or deformation from water pressure acting on the liner plate.

**Table 7** Values of  $(R/t)_c$ 

$(R/t)_c$	$F_y$ , psi (MPa)					
	32,000 (221)	34,000 (234)	35,000 (241)	36,000 (248)	38,000 (262)	40,000 (276)
	377	354	344	334	316	299

### 5.3.5 Meridional compression strength of shells.

5.3.5.1 General. Meridional compression strength  $F_L$  of unstiffened axisymmetric shells shall be determined by Sec. 5.3.5.2. Alternatively, Sec. 5.3.5.3 may be used when the requirements of the section are met. Slenderness effects shall be considered when the ratio of effective length to radius of gyration exceeds 25. Values of  $(R/t)_c$  shall be in accordance with Table 7.

5.3.5.2  $F_L$ : method 1. Meridional compression strength  $F_L$  shall be determined by Eq 5-3a/5-3b. Yield strength  $F_y$  used for design shall not exceed 36,000 psi (248 MPa).

$$F_L = \frac{233}{166 + \frac{R}{t}} F_y \quad \text{not greater than } F_y \quad \text{for } R/t \leq (R/t)_c \quad (\text{Eq 5-3a})$$

$$F_L = \frac{C_o E_s t}{R} \quad \text{for } R/t > (R/t)_c \quad (\text{Eq 5-3b})$$

$C_o$  shall be determined by Eq 5-4.

$$C_o = \frac{102}{195 + \frac{R}{t}} \quad \text{not less than } 0.125 \quad (\text{Eq 5-4})$$

5.3.5.3  $F_L$ : method 2. This section is applicable to water-filled shells that meet the following requirements:

1. Shell configuration where hydrostatic pressure  $p$  causes circumferential tension in the shell.
2. The angle measured from axis of revolution to surface of shell does not exceed 60 degrees.
3. Radius-to-thickness ratio  $R/t$  not greater than 1,000 for shells where plate thickness is controlled by meridional compression strength.
4. Yield strength used in determining meridional compression strength  $F_L$  not greater than 40,000 psi (276 MPa).
5. Minimum plate thickness  $t$  is  $0.8t_{base}$  for  $R/t \leq 700$ , and  $0.7t_{base}$  for  $R/t > 700$ .

6. Tolerance requirements of Sec. 5.4.4.1 are met.

7. Plate elements designed by this section are identified on the construction drawings or QA Plan, and inspection requirements of Sec. 9.4.2.2 are met.

Meridional compression strength  $F_L$  shall be determined by Eq 5-3a/5-3b for  $R/t \leq (R/t)_c$  and by Eq 5-5 for  $R/t > (R/t)_c$ .

$$F_L = F_{cr} \quad \text{not greater than } F_{xe} \quad (\text{Eq 5-5})$$

Critical buckling strength  $F_{cr}$  shall comply with Sec. 5.3.5.4. Maximum meridional compression strength  $F_{xe}$  shall be determined by Eq 5-6.

$$F_{xe} = (C_o + C_p) \frac{\eta E_s t}{R} \quad (\text{Eq 5-6})$$

$C_o$  shall be determined by Eq 5-4 and  $C_p$  shall be determined by Eq 5-7a/5-7b.

$$C_p = \frac{1.06}{3.24 + \frac{E_s t}{f_b R}} \quad \text{for double-curved axisymmetrical and cylindrical shells} \quad (\text{Eq 5-7a})$$

$$= \frac{1.33}{3.33 + \frac{E_s t}{f_b R}} \quad \text{for conical shells} \quad (\text{Eq 5-7b})$$

Determine plasticity reduction factor  $\eta$  by trial using Eq 5-8a/5-8b/5-8c.

$$\eta = 1.0 \quad \text{for } \Delta \leq D_1 \quad (\text{Eq 5-8a})$$

$$\eta = \frac{A}{B + \Delta^{0.8}} \quad \text{for } D_1 < \Delta < D_2 \quad (\text{Eq 5-8b})$$

$$\eta = 1/\Delta \quad \text{for } \Delta \geq D_2 \quad (\text{Eq 5-8c})$$

Buckling coefficient  $\Delta$  shall be determined by Eq 5-9.

$$\Delta = \frac{F_{eff}}{\eta F_y} \quad (\text{Eq 5-9})$$

where 
$$F_{eff} = \sqrt{F_{xe}^2 + F_{xe} f_b + f_b^2} \quad (\text{Eq 5-10})$$

Coefficients  $A$ ,  $B$ ,  $D_1$ , and  $D_2$  shall be determined from Table 8. Circumferential membrane tension stress  $f_b$  in Eq 5-7 and Eq 5-10 shall be based on the hydrostatic pressure at the point under consideration.

5.3.5.4 Critical buckling strength,  $F_{cr}$ . Critical buckling strength  $F_{cr}$  shall be determined from a buckling analysis that complies with the following:

1. Analysis shall be based on numerical solutions using finite element, finite differences, or numerical integration techniques. Effects of material and geometric nonlinearities shall be included.

**Table 8** Plasticity reduction factor coefficients

Coefficient	Yield Strength, $F_y$ , psi (MPa)					
	32,000 (221)	34,000 (234)	35,000 (241)	36,000 (248)	38,000 (262)	40,000 (276)
<i>A</i>	0.773	0.780	0.784	0.788	0.795	0.803
<i>B</i>	0.262	0.256	0.253	0.250	0.244	0.238
<i>D</i> <sub>1</sub>	0.431	0.446	0.453	0.461	0.475	0.490
<i>D</i> <sub>2</sub>	5.100	4.870	4.760	4.660	4.460	4.270

2. Analysis shall consider gross structural discontinuities; including shell discontinuity junctures, changes in plate thickness, plate misalignment; and imperfections. Modeled imperfection shall be sinusoidal or similar shape having a length  $L_x$  and imperfection ratio  $e_x/L_x$  not less than the construction tolerances permitted in Sec. 5.4.4.1, and shall be located to produce the lowest critical buckling strength  $F_{cr}$ .

3. The modeled boundary conditions shall result in displacements and rotations compatible to those of the actual structure.

4. Load level at which the structure becomes unstable shall be determined by applying service loads followed by loads required to cause instability. Critical failure load may be determined by incremental increase in meridional force, incremental increase in fluid density, or both. Hydrostatic pressure shall not exceed the pressure at operating conditions.

5. Use a stress-strain curve that includes the effects of residual stresses due to fabrication and welding; or use a stress-strain curve that includes residual stresses due to fabrication and include the effects of welding by introducing equivalent strains or stresses to the model. Alternatively, account for the effects of residual stresses by using modeled imperfection ratio  $e_x/L_x$  not less than twice the construction tolerances permitted in Sec. 5.4.4.1.

6. Critical buckling strength  $F_{cr}$  shall be determined for each shell course of different thickness. Analysis shall be based on the thickness of each shell course less the specified corrosion allowance.

## Sec. 5.4 Fabrication and Construction Requirements

### 5.4.1 Weld procedure and welder qualification.

5.4.1.1 Weld procedure specifications. All welding shall be in accordance with written weld procedure specifications (WPS) complying with ASME Section IX or AWS B2.1.

5.4.1.2 Welder/welding operator qualification. Welders and welding operators shall be qualified to the same code as the weld procedure specifications.

5.4.2 *Fabrication and erection.*

5.4.2.1 Plate forming and cutting. Steel plate and components shall be cut and formed to meet the tolerance requirements of Sec. 5.4.4.

5.4.2.2 Preparation of base metal. Preparation of base metal shall comply with Sec. 5.15 of AWS D1.1.

5.4.2.3 Weld consumables and electrodes. Material, handling, and storage of weld consumables and electrodes shall comply with Sec. 5.3 of AWS D1.1.

5.4.2.4 Alignment and fit of weld joints. Parts joined by welding shall be brought into correct alignment and held in position by suitable devices or by tack welds until welding has been completed. Allowance shall be made for distortion from welding. Root opening of fillet welds shall not exceed  $\frac{3}{16}$  in. (4.76 mm). Fillet weld leg size shall be increased by the amount of root opening when it exceeds  $\frac{1}{16}$  in. (1.59 mm).

5.4.3 *Welding operation.*

5.4.3.1 Weld quality. Welding shall meet the quality requirements of Sec. 9.5 and shall meet the tolerance requirements of Sec. 5.4.4.2.

5.4.3.2 Weld joint records. A record shall be kept that identifies which welder or welding operator performed welding on each shell joint.

5.4.3.3 Welding environment. Protect the area in the immediate vicinity of the weld from wind, moisture, low temperature, and other conditions that may adversely affect quality of the weld. Maintain environment temperature adjacent to the weldment above 0°F (-20°C). Alternatively, when the base metal temperature is below 0°F (-20°C), the base metal along the length of the weld joint in the direction of welding shall be preheated and maintained in accordance with Sec. 5.4.3.6 as welding progresses.

5.4.3.4 Temporary and tack welds. Temporary welds and tack welds not incorporated into final welds shall be removed. The weld procedure for final welds applies to temporary and tack welds incorporated into final welds.

5.4.3.5 Control of distortion and shrinkage. Joints shall be welded in a manner to minimize distortion from weld shrinkage.

5.4.3.6 Preheat and interpass temperature. Preheat and interpass temperature shall comply with the following:

1. Preheat and maintain interpass temperatures in accordance with weld procedure specifications.

2. Preheat to 70°F (21°C) when the base metal temperature is below 32°F (0°C), unless higher preheat and interpass temperatures are required by the weld procedure specifications.

3. For material thickness greater than 1.5 in., preheat and interpass temperature shall not be less than 100°F (38°C) when low-hydrogen electrode is used, and 200°F (93°C) when other than low-hydrogen electrode is used.

4. Apply preheat to an area of base metal at least 3 in. (76 mm) beyond the limits of welding.

5. Measure preheat temperature just prior to welding.

6. For combinations of material thickness, minimum preheat shall be based on the highest minimum preheat.

5.4.3.7 Weld tabs. Weld tabs shall be used when necessary to provide an extension of the joint preparation and shall be removed on completion of the weld.

5.4.3.8 Weld repair. Machining, grinding, chipping, or gouging shall be used to remove unacceptable portions of welds without removal of substantial base metal. Repair welding shall comply with weld procedure specifications. Weld metal shall be deposited to compensate for any deficiency in size or undercut.

5.4.3.9 Weld cleaning. Remove all slag and brush clean adjacent base metal before welding over previously deposited metal. After completion of welding, remove all slag, temporary welds, weld spatter, and blemishes caused by arc strikes.

#### 5.4.4 Tolerances.

5.4.4.1 Local imperfections. Deviation from the theoretical shape shall not exceed the plate flatness and waviness tolerances permitted in ASTM A6 and ASTM A20, whichever is applicable for the material specified. For shell elements designed using Sec. 5.3.5, imperfection ratio  $e_x/L_x$  shall not exceed 0.01. Gauge length  $L_x$  shall be  $4\sqrt{Rt}$  along the meridian.

5.4.4.2 Welded joints. Welded joints shall comply with the following:

1. Imperfection at butt joints. Out-of-plane distortion across a meridional or circumferential weld seam shall not exceed ½ in. (12.7 mm) when measured with a 36-in. (0.91-m) long profile board.

2. Alignment of butt joints. Alignment of adjoining edges of butt joints shall comply with Table 9.

3. Weld profiles. Weld profiles shall comply with Sec. 5.24 of AWS D1.1, except weld reinforcement of shell butt joints shall comply with Table 10. Undercut shall not exceed the following:

**Table 9 Alignment tolerance of butt joints**

Shell Plate Thickness $t^*$		Primary Tension Stress Joint		Other Joints	
(in.)	(mm)	(in.)	(mm)	(in.)	(mm)
$0 < t \leq \frac{3}{8}$	$0 < t \leq 15.9$	$\frac{1}{16}$	1.59	$\frac{1}{8}$	3.18
$> \frac{3}{8}$	$> 15.9$	lesser of $0.10t$ or $\frac{1}{4}$	lesser of $0.10t$ or $\frac{1}{4}$	lesser of $0.20t$ or $\frac{3}{8}$	lesser of $0.20t$ or 9.52

\* Thickness of the thinner plate at the joint.

**Table 10 Maximum reinforcement of shell butt joints\***

Shell Plate Thickness $t$		Primary Tension Stress Joint		Other Joints	
(in.)	(mm)	(in.)	(mm)	(in.)	(mm)
$t \leq \frac{1}{2}$	$t \leq 12.7$	$\frac{3}{32}$	2.38	$\frac{1}{8}$	3.18
$\frac{1}{2} < t \leq 1$	$12.7 < t \leq 25$	$\frac{1}{8}$	3.18	$\frac{3}{16}$	4.76
$t > 1$	$t > 25$	$\frac{3}{16}$	4.76	$\frac{1}{4}$	6.35

\* Excess reinforcement need only be removed to the extent to which it exceeds the allowable tolerance.

a. Primary tension stress joint  $\frac{1}{64}$  in. (0.40 mm)

b. Other joints  $\frac{1}{32}$  in. (0.80 mm)

5.4.5 *Painting.* Unless otherwise specified, painting of the steel tank shall comply with ANSI/AWWA D102.

5.4.6 *Disinfecting.* Disinfection of the steel tank shall be in accordance with ANSI/AWWA C652.

## SECTION 6: CONCRETE SUPPORT STRUCTURE

### Sec. 6.1 General

6.1.1 *Scope.* This section covers materials, design, and construction of the concrete support structure.

6.1.2 *Notation.* The following symbols are used to represent variables in this section:

$A_{cv}$  = shear wall area resisting factored tangential shear force  $V_{uw}$ , in.<sup>2</sup>

$A_s$  = area of tension reinforcement, in.<sup>2</sup>

$A_v$  = effective peak-velocity-related acceleration coefficient



- $A_w$  = gross horizontal cross-sectional area of support wall based on thickness  $b_w$ , in.<sup>2</sup> per unit of circumference  
 $b$  = width of compression face of concrete member, in.  
 $B_d$  = width of support wall opening, in.  
 $B_e$  = effective column width adjacent to support wall opening, in.  
 $B_x$  = cumulative width of openings in the effective shear wall width  $0.78D_w$ , in.  
 $c_c$  = clear cover from nearest surface in tension to surface of flexural tension reinforcement, in.  
 $d$  = distance from extreme compression fiber to centroid of tension reinforcement, in.  
 $D_w$  = mean diameter of support wall, in.  
 $f_c'$  = specified compressive strength of concrete, psi  
 $f_s$  = service load stress in tension reinforcement, psi  
 $f_y$  = specified yield strength of reinforcement, psi  
 $h_d$  = dome tank floor thickness, in.  
 $h_w$  = wall thickness exclusive of any architectural relief, in.  
 $H_d$  = height of support wall opening, in.  
 $k_l$  = effective unsupported column length of support wall opening  
 $M_h$  = unfactored circumferential wind moment, lb-in. per unit of height  
 $M_u$  = factored moment, lb-in.  
 $p_z$  = wind pressure at height  $z$  above ground level, lb/in.<sup>2</sup>  
 $P_{nw}$  = nominal axial compression strength of support wall, lb per unit of circumference  
 $P_{uw}$  = factored axial load on support wall, lb per unit of circumference  
 $V_n$  = nominal tangential shear strength, lb  
 $V_u$  = factored shear force, lb  
 $V_{uw}$  = factored tangential shear force acting on an effective shear wall, lb  
 $R_d$  = mean spherical radius of dome tank floor, in.  
 $S$  = maximum spacing of flexural or tension reinforcement closest to a surface, in.  
 $z$  = height above ground level, ft  
 $\alpha_c$  = constant used to calculate nominal tangential shear strength  $V_n$   
 $\beta_w$  = local slenderness coefficient for the support wall  
 $\rho$  = ratio of area of tension reinforcement  $A_s$  to area  $bd$   
 $\rho_g$  = ratio of area of total longitudinal reinforcement to gross cross-sectional area of member

- $\rho_n$  = ratio of area of distributed shear reinforcement to shear wall area  $A_{cv}$   
 $\phi$  = strength reduction factor  
 $\Psi$  = wall opening ratio

## Sec. 6.2 Material Requirements

6.2.1 *General.* Materials for structural concrete shall comply with the requirements of ACI 318, except as modified by this standard.

6.2.2 *Cement.* The same brand, type, and source of cement shall be used throughout the construction of each major element.

6.2.3 *Aggregates.* Concrete aggregates shall conform to ASTM C33.

6.2.4 *Admixtures.* Admixtures containing calcium chloride or other chlorides shall not be used for structural concrete.

## Sec. 6.3 Analysis and Design

6.3.1 *Analysis.* Analysis of the concrete support structure shall be in accordance with ACI 318. The support wall and dome tank floor shall be analyzed in accordance with chapter 19 of ACI 318.

6.3.2 *General design requirements.*

6.3.2.1 *Design method.* Design the concrete support structure by the strength design method in accordance with ACI 318 to resist the factored load combinations in Sec. 4.3.4.

6.3.2.2 *Minimum flexural reinforcement.* Where flexural reinforcement is required by analysis, the reinforcement ratio  $\rho$  shall not be less than  $3\sqrt{f'_c}/f_y$  nor  $200/f_y$ . 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.

6.3.2.3 *Minimum axial tension reinforcement.* In regions of axial tension, the reinforcement ratio  $\rho_g$  shall not be less than  $5\sqrt{f'_c}/f_y$ . 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.

6.3.2.4 *Spacing of tension reinforcement.* At cross sections where flexural or axial tension reinforcement is required by analysis, reinforcement spacing shall comply with the following for load combination S1.1 in Sec. 4.3.3.1:

1. Spacing  $S$  of flexural reinforcement closest to a surface in tension shall not exceed:

$$S = 15 \left( \frac{40,000}{f_s} \right) - 2.5c_c \leq 12 \left( \frac{40,000}{f_s} \right) \quad (\text{Eq 6-1})$$

2. Spacing of axial tension reinforcement closest to a surface shall not exceed 80 percent of the spacing determined by Eq 6-1.

6.3.3 *Support wall design.*

6.3.3.1 Design requirements. The concrete support wall shall be cylindrical and shall be designed for maximum loads using theory of elastic shell analysis and gross section properties. Maximum specified compressive strength of concrete  $f'_c$  used for design shall be 6,000 psi (41 MPa).

6.3.3.2 Minimum wall thickness. Wall thickness  $h_w$  shall not be less than 8 in. (200 mm). Compression stress shall not exceed  $0.25f'_c$  nor 1,000 psi (7 MPa) for load combination S1.1 in Sec. 4.3.3.1.

6.3.3.3 Reinforcement. Wall reinforcement shall be placed in two layers. Not less than 50 percent nor more than 60 percent of the minimum reinforcement shall be distributed to the exterior face. Transverse reinforcement conforming to Sec. 7.10.5 of ACI 318 shall be provided at sections requiring compression reinforcement.

Minimum reinforcement for tangential shear shall comply with Table 11 and Sec. 6.3.3.8.2.

6.3.3.4 Axial load and flexure. Tensile strength of concrete shall be neglected in strength calculations for direct tension or direct tension combined with flexure. Walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with Sec. 10.2, 10.3, and 10.10 of ACI 318. The effects of flexure may be neglected at sections where the resultant of the vertical load is within the middle third of the wall cross section. Design for axial load shall be based on

$$P_{uw} \leq \phi\beta_w P_{nw} \tag{Eq 6-2}$$

**Table 11 Wall reinforcement requirements**

Reinforcement	Minimum Reinforcement Ratio
Vertical reinforcement:	
No. 11 (36) bar and smaller, and plain and deformed welded wire fabric	0.0015
Horizontal reinforcement:	
No. 5 (16) bar and smaller, and plain and deformed welded wire fabric	0.0020
No. 6 (19) bar and larger	0.0025

The strength reduction factor  $\phi$  shall be 0.65. The local slenderness coefficient  $\beta_w$  shall be determined from Eq 6-3 or by a more detailed analysis. The coefficient  $\beta_w$  shall not be greater than 1.0.

$$\beta_w = 80 \frac{h_w}{D_w} \quad (\text{Eq 6-3})$$

6.3.3.5 Axial compression strength. Nominal axial compression strength  $P_{nw}$  shall not exceed the following:

1. At sections where transverse tie reinforcement is not provided:

$$P_{nw} = 0.55f'_c A_w \quad (\text{Eq 6-4a})$$

2. At sections where transverse tie reinforcement is provided:

$$P_{nw} = [0.68f'_c + 0.8(f_y - 0.85f'_c)\rho_g]A_w \quad (\text{Eq 6-4b})$$

6.3.3.6 Circumferential bending moment. Horizontal reinforcement shall be provided in each face to resist circumferential moments arising from variations in wind pressure over the wall circumference. Except where a more detailed analysis is performed, the unfactored wind moment  $M_h$  is

$$M_h = 0.05p_x D_w^2 \quad (\text{Eq 6-5})$$

The wind pressure  $p_x$  at height  $z$  above ground level shall be in accordance with Sec. 4.2.6. The moment  $M_h$  may be considered to vary linearly from zero at a diaphragm or other effective restraint to the full value at a distance  $0.5D_w$  from the diaphragm.

6.3.3.7 Radial shear. Design of the concrete support structure for radial shear shall comply with chapter 11 of ACI 318.

6.3.3.8 Tangential shear. Design for tangential shear shall be based on

$$V_{uw} \leq \phi V_n \quad (\text{Eq 6-6})$$

Factored tangential shear force  $V_{uw}$  shall not exceed  $3\sqrt{f'_c}A_{cv}$ .

The strength reduction factor  $\phi$  shall be 0.75. Where tangential shear is due to seismic loads, the strength reduction factor  $\phi$  shall be 0.60.

6.3.3.8.1 Tangential shear force. A detailed analysis for tangential shear shall be used when the wall opening ratio  $\psi$  in Eq 6-10 is greater than 0.5. Lateral loads shall be distributed to the lateral-load-resisting elements in proportion to their relative stiffness.

6.3.3.8.2 Tangential shear strength. The nominal tangential shear strength  $V_n$  shall not exceed

$$V_n = (\alpha_c \sqrt{f'_c} + \rho_n f_y) A_{cv} \quad (\text{Eq 6-7})$$

where:

$$\alpha_c = 6 - \frac{2.5M_u}{V_u D_w} \quad 2.0 < \alpha_c \leq 3.0 \quad (\text{Eq 6-8})$$

$M_u$  and  $V_u$  are the total factored moment and shear occurring simultaneously at the section under consideration. The ratio of distributed shear reinforcement  $\rho_n$  used for design shall be based on the smaller of horizontal or vertical reinforcement areas.

When tangential shear  $V_{uw}$  exceeds  $\sqrt{f'_c} A_{cv}$ , minimum horizontal and vertical reinforcement ratios shall not be less than 0.0025.

Unless a detailed analysis is performed, concrete shear area  $A_{cv}$  shall not exceed

$$A_{cv} = 0.78 (1 - \psi) D_w h_w \quad (\text{Eq 6-9})$$

The wall opening ratio  $\psi$  shall be

$$\Psi = \frac{B_x}{0.78 D_w} \quad (\text{Eq 6-10})$$

Nominal tangential shear strength  $V_n$  shall be determined at a distance above the foundation equal to the smaller of  $0.39D_w$  or the distance from the foundation to mid-height of the largest opening.

#### 6.3.4 Dome tank floor design.

6.3.4.1 Design requirements. Concrete dome tank floors shall terminate in a ringbeam, and shall be designed on the basis of elastic shell analysis. Consideration of edge effects that cause shear and moment shall be included in the analysis and design. Maximum specified compressive strength of concrete  $f'_c$  used for design shall be 5,000 psi (34 MPa).

6.3.4.2 Dome. Dome tank floor thickness  $h_d$  shall not be less than 8 in. (200 mm). Compression stress shall not exceed  $0.13f'_c$  nor 600 psi (4.1 MPa) for load combination S1.1 in Sec. 4.3.3.1. Buckling effects shall be considered when the radius-to-thickness ratio  $R_d/h_d$  exceeds 100. At cross sections not governed by Sec. 6.3.2.2 or Sec. 6.3.2.3, the minimum reinforcement area in each face shall be 0.0018 times the gross concrete area in orthogonal directions.

6.3.4.3 Ringbeam. Compression stress due to unbalanced horizontal thrust forces and edge effects shall not exceed  $0.18f'_c$  nor 800 psi (5.5 MPa) for load combination S1.1 in Sec. 4.3.3.1. Minimum circumferential reinforcement area shall be 0.004 times the gross concrete area. Transverse reinforcement shall be

No. 4 bar size or larger. Spacing shall not exceed 16 longitudinal bar diameters or 18 in. (460 mm).

**6.3.5 Slab tank floor design.** Maximum specified compressive strength of concrete  $f'_c$  used for design shall be 5,000 psi (34 MPa). Flexural stiffness of the slab shall be sufficient to prevent excessive deformation of the attached wall and steel tank elements under load combination S1.1 in Sec. 4.3.3.1.

**6.3.6 Interface region design.** Details to divert rain water and condensation from the interface region shall be incorporated into the design. Provide a design that considers the following:

1. Interaction of loads from the steel tank, tank floor, and support wall.
2. Loading resulting from varying water level.
3. Unsymmetrical loading conditions from seismic and wind forces.
4. Construction loads and attachments.
5. 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.
6. Eccentricity of loads, where the point of application of load does not coincide with the centroid of the resisting elements.
7. Effect of restrained shrinkage and temperature differentials.
8. Anchorage attachments when required for uplift loads.
9. Local instability of the top of the support wall when stiffening elements are not provided.
10. Effect of construction tolerances.

**6.3.7 Design of wall openings.**

**6.3.7.1 General.** Openings may be designed in accordance with Sec. 6.3.7.2 or Sec. 6.3.7.3 when the following conditions are met:

1. The offset between chord and arc across the opening is 1.5 ft (0.46 m) or less.
2. The clear distance between openings is 0.75 times the cumulative width of adjacent openings or greater.
3. The cumulative width of openings is less than 20 percent of the support wall circumference.

Otherwise, design of openings shall be based on a detailed analysis that considers the three-dimensional geometry of the openings and the effect of applied vertical and lateral loads.

**6.3.7.2 Small openings.** Isolated openings or penetrations having a maximum dimension of 2 ft (0.61 m) do not require analysis. Additional reinforcement

having an area not less than 1.2 times the area of interrupted reinforcement shall be equally distributed to either side of the opening.

6.3.7.3 Effective beam and column method. Wall openings may be designed by the following approximate procedure:

1. Each side of the opening shall be designed as a reinforced concrete column in accordance with ACI 318. Effective column width  $B_e$  shall not exceed  $(2 + B_d/48)h_w$ . The column design load shall be the factored axial load  $P_{uw}$  acting at mid-height of the opening times  $(B_e + 0.5B_d)$ .

2. The effective unsupported column length  $kl$  shall not be less than  $0.85H_d$ .

3. Horizontal reinforcement with an area not less than determined from Eq 6-11 shall be provided above and below the opening. The reinforcement shall be distributed within a height  $3h_w$ .

$$A_s = \frac{0.14P_{uw}B_d}{\phi f_y} \quad (\text{Eq 6-11})$$

The strength reduction factor  $\phi$  shall be 0.9 for tension.

6.3.7.4 Pilasters. Pilasters built monolithically on the interior of the support wall may be used adjacent to openings. The transition zone where pilasters are terminated shall be analyzed and reinforced for local stresses. Pilasters shall be designed as tied columns in accordance with ACI 318.

6.3.7.5 Development of reinforcement. Reinforcement provided for openings shall be fully developed. Horizontal reinforcement shall project at least half a development length beyond the longitudinal column reinforcement.

6.3.7.6 Vehicle impact. Pipe bollards shall be provided at vehicle access openings as protection for vehicle impact.

## Sec. 6.4 Fabrication and Construction Requirements

6.4.1 *General.* Construction of reinforced concrete shall comply with the requirements of ACI 318, except as modified by this standard.

6.4.2 *Concrete mixtures.*

6.4.2.1 Concrete quality. Concrete shall comply with the durability requirements of ACI 318 and the following:

1. The water-cementitious material ratio shall not exceed 0.50.
2. Minimum specified compressive strength of concrete  $f'_c$  shall be 4,000 psi (28 MPa).\*

\* See Sec. 6.3.3.1, Sec. 6.3.4.1, and Sec. 6.3.5 for maximum specified strengths.

3. Unless otherwise specified, concrete shall be air-entrained for moderate exposure in accordance with ACI 318.

6.4.2.2 Proportioning. Concrete shall be proportioned in accordance with ACI 318 on the basis of field experience or trial mixtures.

6.4.3 *Details of reinforcement.*

6.4.3.1 Fabrication. Fabrication of reinforcement shall comply with the requirements of ACI 318.

6.4.3.2 Concrete cover. Minimum concrete cover shall be in accordance with ACI 318 but not less than 1 in. (25 mm).

6.4.3.3 Supports. Support reinforcement adjacent to formwork with metal or plastic bar supports. The portions of bar supports within ½ in. (13 mm) of the concrete surface shall be noncorrosive or protected against corrosion.

6.4.4 *Curing and protection.*

6.4.4.1 General. Protect concrete from premature drying, excessively hot or cold temperatures, and mechanical injury. The selected curing methods shall be continued until the concrete attains 70 percent of the specified compressive strength. Curing compounds shall conform to ASTM C309.

6.4.4.2 Cold weather protection. Protect concrete in cold weather in accordance with ACI 306.1. Protection against freezing may be discontinued when the concrete has attained a compressive strength of 1,000 psi (7 MPa), unless a higher strength is required for the applied loads.

6.4.4.3 Hot weather protection. Protect concrete from high temperature, direct sunlight, low humidity, and drying winds after placing and finishing operations are complete.

6.4.5 *Formwork.*

6.4.5.1 General. Formwork shall comply with the following:

1. The contact surface of forms used above grade shall be metal, plastic, or fiberglass.

2. Exposed corners shall be formed with chamfers ¾ in. (19 mm) or larger.

3. Form surfaces shall be clean and free of foreign materials. Form coating materials shall not be applied to reinforcement or hardened concrete against which fresh concrete is to be placed.

4. Formwork may be removed when the concrete has sufficient strength to prevent damage from the removal operation and subsequent loads. The minimum compressive strength of concrete required for safe removal of forms or supports



and use of construction embedments or attachments shall be identified on the construction drawings.

6.4.5.2 Wall forms. Wall forms shall comply with the following:

1. The support wall form system shall consist of curved, prefabricated form segments, ties, bracing, and supports. Formwork shall be designed for full-height plastic concrete head lateral pressure, construction loads, and wind loads. Provide working platforms for inspection and concrete placement.

2. Calculated deflection of facing material and supporting elements shall not exceed the larger of  $\frac{1}{360}$  times the span or  $\frac{1}{16}$  in. (1.6 mm).

3. Provide a uniform pattern of vertical and horizontal rustications for architectural relief on exterior wall surfaces exposed to view. Construction joints shall be located in rustications.

6.4.5.3 Tank floor forms. Formwork for concrete tank floors shall be designed to support construction loads. Unsymmetrical placement of concrete shall be considered in the design. Provide camber where required to meet tolerances.

6.4.6 Tolerances. Tolerances for concrete construction shall comply with ACI 117 and the following:

1. Support wall variation:

Thickness	-3 percent, +5 percent
Diameter	0.4 percent $\leq$ 3 in. (76 mm)
Vertical alignment in any 10 ft (3.0 m) of height	1 in. (25 mm)
Vertical alignment in any 50 ft (15.2 m) of height	2 in. (50 mm)
Vertical alignment over total height	3 in. (76 mm)

2. Tank floor variation:

Slab floor thickness	-3 percent, +5 percent
Dome floor thickness	-6 percent, +10 percent
Dome floor normal radius	1 percent
Finish tolerance measured with a 5-ft (1.5-m) straight edge or radius board	$\frac{3}{4}$ in. (19 mm)

3. Level alignment variation:

From specified elevation	1 in. (25 mm)
From horizontal plane	$\frac{1}{2}$ in. (13 mm)

## 4. Offset between adjacent forms:

Exterior exposed surfaces	1/8 in. (3 mm)
Interior exposed surfaces	1/4 in. (6 mm)

6.4.7 *Finishes.*

6.4.7.1 Formed surfaces. Remove fins, and repair surface defects greater than 3/8-in. (10-mm) depth. Tie holes shall be patched, except that manufactured plastic plugs may be used for exterior surfaces. Provide additional finishing as specified.

6.4.7.2 Unformed surfaces. Unless otherwise specified, unformed surfaces shall have a floated finish.

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## SECTION 7: FOUNDATIONS

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**Sec. 7.1 General**

7.1.1 *Scope.* This section covers materials, design, and construction of the foundation.

7.1.2 *Notation.* The following symbols are used to represent variables in this section:

$A_s$  = area of tension reinforcement, in.<sup>2</sup>

$b$  = width of compression face of concrete member, in.

$d$  = distance from extreme compression fiber to centroid of tension reinforcement, in.

$f'_c$  = specified compressive strength of concrete, psi

$f_{pc}$  = effective prestress on the gross section of precast prestressed concrete piles, psi

$f_s$  = steel stress due to unfactored loads, psi

$f_y$  = yield strength of reinforcement, psi

$F_y$  = minimum specified yield strength of structural steel and steel-cased piles, psi

$\rho$  = ratio of area of tension reinforcement  $A_s$  to area  $bd$

7.1.3 *Geotechnical investigation.* A geotechnical investigation shall be performed to determine the engineering properties of the supporting soil or rock for foundation design. Appendix C provides recommendations for conducting the geotechnical investigation.

**Sec. 7.2 Material Requirements****7.2.1 Concrete.**

7.2.1.1 General. Materials for structural concrete shall comply with the requirements of ACI 318, except as modified by this standard.

7.2.1.2 Aggregates. Concrete aggregates shall conform to ASTM C33.

7.2.1.3 Admixtures. Admixtures containing calcium chloride or other chlorides shall not be used for structural concrete.

7.2.2 Piles. Unless otherwise specified, pile types and pile materials shall be in accordance with the recommendations of ASCE 20. The manufacture of pre-cast and prestressed concrete piles shall comply with PCI MNL 116.

**Sec. 7.3 Analysis and Design**

7.3.1 *Sizing foundations.* Determination of base areas of shallow foundations and number and arrangement of piles for deep foundations shall be based on principles of soil mechanics and/or load testing, and shall satisfy requirements for strength, settlement, overturning stability, and lateral resistance.

7.3.1.1 Strength. Foundations shall be sized using the unfactored load combinations in Sec. 4.3.3. Allowable bearing capacity for shallow foundations and allowable pile capacity for deep foundations shall not exceed ultimate capacity divided by the minimum factor of safety in Table 12. Unless otherwise specified, a one-third increase in allowable bearing capacity and allowable pile capacity is permitted for load combinations that include wind or seismic loads.

**Table 12 Minimum factor of safety for foundations**

Foundation Type and Method of Establishing Ultimate Capacity	Load Combinations S1.1 and S1.2	Load Combinations S1.3 and S1.4*
Shallow Foundations:		
Analysis using engineering principles	3.0	2.25
Deep Foundations:		
Analysis using engineering principles	3.0	2.25
High-strain dynamic testing of driven piles in accordance with ASTM D4945	2.25	1.69
Static load testing in accordance with ASTM D1143, and high-strain dynamic testing of driven piles in accordance with ASTM D4945 using signal matching analysis or other in-situ load tests that determine end bearing, side friction, or both	2.0	1.5

\* The factors of safety for load combinations S1.3 and S1.4 reflect a one-third increase in allowable load for load combinations that include wind or seismic loads in accordance with Sec. 7.3.1.

7.3.1.2 Settlement. Foundations shall be sized to limit total and differential settlement to values that do not cause distress or undue distortion to the structure, connecting piping, or accessories. Tilting shall not exceed  $\frac{1}{300}$ .

7.3.1.3 Overturning stability. Foundations shall be sized to resist overturning forces resulting from wind, seismic forces, and differential settlement. The ratio of the resisting moment to overturning moment shall be 1.5 or greater for Group 2 load combinations defined in Sec. 4.3.3.2. Resisting moment shall be calculated about the center of gravity of the vertical loads.

7.3.1.4 Lateral resistance. Foundations shall be sized to resist lateral loads resulting from wind and seismic forces. Friction, passive soil resistance, or both may be used to determine lateral resistance of shallow foundations. Passive soil resistance, lateral pile resistance, or both may be used to determine lateral resistance of deep foundations.

### 7.3.2 *Structural design of concrete foundations.*

7.3.2.1 General. Design shallow foundations or pile caps by the strength design method in accordance with ACI 318 to resist the factored load combinations of Sec. 4.3.4. Maximum specified compressive strength  $f'_c$  shall be 5,000 psi (34 MPa). The effect of pile plan location tolerance shall be included in the design of pile caps.

7.3.2.2 Minimum flexural reinforcement. Where flexural reinforcement is required by analysis, the minimum reinforcement ratio  $\rho$  shall not be less than  $3\sqrt{f'_c}/f_y$  nor less than  $200/f_y$ . 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.

7.3.2.3 Serviceability. At sections of maximum moment in shallow foundations and pile caps, the tension stress in reinforcement  $f_s$  shall not exceed 30,000 psi (205 MPa) for load combination S1.1. Alternatively, spacing of tension reinforcement at sections of maximum moment may be designed in accordance with Sec. 6.3.2.4.

7.3.2.4 Support wall openings. The effect on the foundation of openings in the support wall may be neglected when the distance from bottom of opening to top of foundation is greater than one-half the opening width.

### 7.3.3 *Structural design of piles.*

7.3.3.1 General. Piles shall be designed for axial and lateral loads and moments due to handling, installation, and the load combinations in Sec. 4.3.3. Design shall be in accordance with requirements of applicable building codes for

the type of pile material. Maximum specified compressive strength  $f'_c$  of cast-in-place concrete and grout shall be 5,000 psi (34 MPa).

7.3.3.2 Allowable stress. Allowable stress in piles shall not exceed the values permitted by the project specification or applicable building code for unfactored load combinations in Sec. 4.3.3. Allowable stress may be increased by one-third for load combinations that include wind or seismic loads.

7.3.3.3 Seismic design details. Design and detailing of piles subject to seismic forces shall comply with ASCE 7.

#### 7.3.4 Foundation details.

7.3.4.1 Minimum foundation depth. Unless otherwise specified, minimum foundation depth shall be determined from Figure 1. Minimum foundation depth may be reduced if the foundation bears on sound rock. Pipe cover specified in Sec. 8.7.7 shall be considered when it affects determination of foundation depth.

7.3.4.2 Grading and drainage. Potential settlements shall be considered when selecting grading and paving elevations. Provide positive drainage away from the structure to prevent ponding of water in the foundation area.

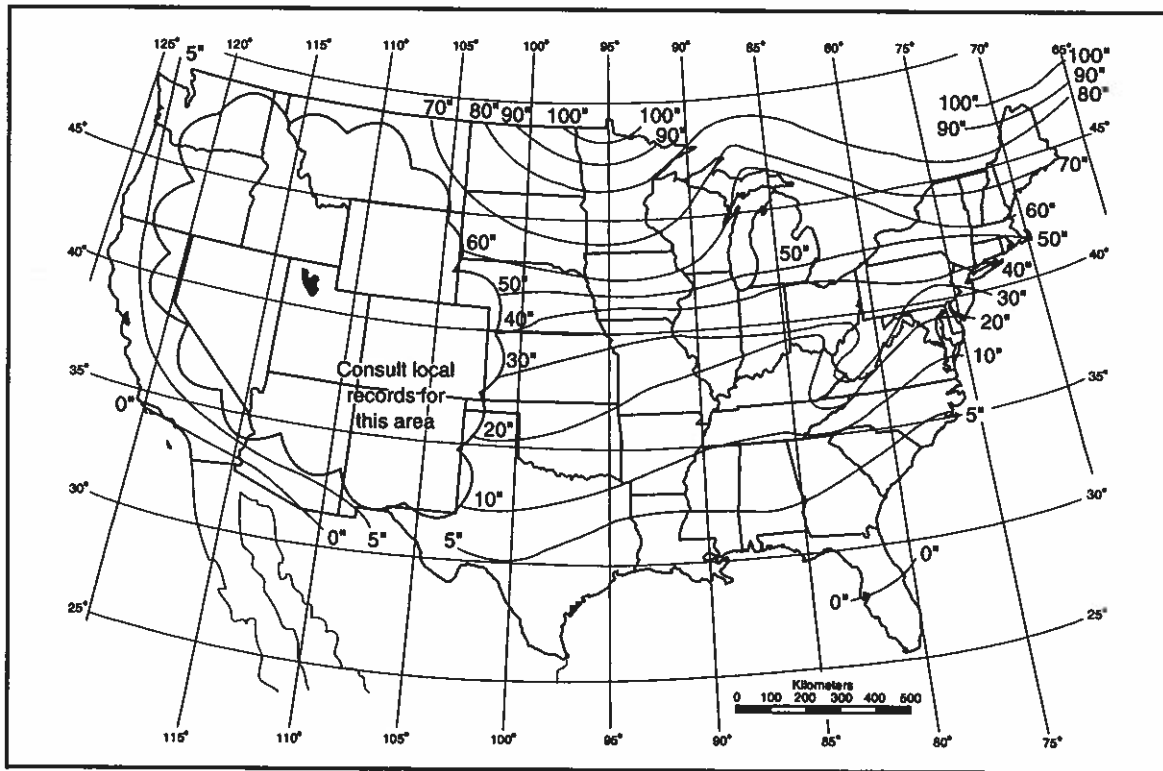


Figure 1 Extreme frost penetration (depth in inches based on state averages)



7.3.4.3 **Pile caps.** Pile forces and moments shall be transferred to pile caps by embedment, bearing on concrete, or reinforcement. Pile caps shall extend horizontally beyond the edge of piles a sufficient distance to develop tension reinforcement but not less than 4 in. (100 mm). Provide minimum 3-in. (76-mm) embedment. For a single row of piles with minimum 36-in. (910-mm) width, pile cap width may be less than the pile width and minimum embedment may be reduced to zero.

7.3.4.4 **Piles.** Spacing of individual piles shall be based on evaluation of axial and lateral load capacity. Splices shall be designed for loads occurring at the connection.

## **Sec. 7.4 Fabrication and Construction Requirements**

### **7.4.1 Earthwork.**

7.4.1.1 **Excavation.** Excavations shall comply with applicable safety standards and regulations. The bearing surface shall be unfrozen and free of standing water and deleterious material when foundation concrete is placed. Bearing surfaces subject to deterioration shall be protected with a layer of concrete.

7.4.1.2 **Backfill.** Soils used for fill shall be unfrozen and free of organic matter and debris. Unless otherwise specified, provide the following:

1. Within the concrete support wall backfill with suitable material, and compact to 95 percent standard Proctor density (ASTM D698).
2. Outside the concrete support wall backfill with unclassified soils, and compact to 90 percent standard Proctor density (ASTM D698).

### **7.4.2 Concrete foundations.**

7.4.2.1 **General.** Construction of reinforced concrete shall comply with the requirements of ACI 318, except as modified by this standard.

#### **7.4.2.2 Concrete mixtures.**

7.4.2.2.1 **Concrete quality.** Concrete shall comply with the durability requirements of ACI 318 and the following:

1. Water-cementitious material ratio shall not exceed 0.50.
2. Minimum specified compressive strength of concrete for shallow foundations and pile caps shall be 3,500 psi (24 MPa).
3. Unless otherwise specified, concrete exposed to freeze-thaw cycles shall be air-entrained for moderate exposure in accordance with ACI 318.

7.4.2.2.2 **Proportioning.** Concrete shall be proportioned in accordance with ACI 318 on the basis of field experience or trial mixtures.

### 7.4.2.3 Details of reinforcement.

7.4.2.3.1 Fabrication. Fabrication of reinforcement shall comply with the requirements of ACI 318.

7.4.2.3.2 Concrete cover. Minimum concrete cover shall be in accordance with ACI 318 but not less than 1½ in. (38 mm).

### 7.4.2.4 Curing and protection.

7.4.2.4.1 General. Protect concrete from premature drying, excessively hot or cold temperatures, and mechanical injury. The selected curing methods shall be continued until the concrete attains 70 percent of the specified compressive strength. Curing compounds shall conform to ASTM C309.

7.4.2.4.2 Cold weather protection. Protect concrete in cold weather in accordance with ACI 306.1. Protection against freezing may be discontinued when the concrete has attained a compressive strength of 1,000 psi (7 MPa), unless a higher strength is required for applied loads.

7.4.2.4.3 Hot weather protection. Protect concrete from high temperature, direct sunlight, low humidity, and drying winds after placing and finishing operations are complete.

7.4.2.5 Formwork. Formwork shall comply with the following requirements:

1. Exposed corners shall be formed with chamfer ¾ in. (19 mm) or larger.
2. Form surfaces shall be clean and free of foreign material. Form coating materials shall not be applied to reinforcement or hardened concrete against which fresh concrete is to be placed. Straight form segments that circumscribe the design radius may be used to form circular shapes.
3. The included angle of any segment shall not exceed 30 degrees.
4. Top forms shall be provided for surfaces steeper than 2.5 (horizontal) to 1 (vertical), unless it can be demonstrated that the required shape can be maintained during placement and consolidation.
5. Formwork may be removed when the concrete has sufficient strength such that the removal operation and subsequent loads will not cause damage. The minimum compressive strength of concrete required for safe removal of forms or supports and use of construction embedments or attachments shall be identified on the construction drawings.

7.4.2.6 Concrete foundation tolerances. Tolerances for shallow foundations and pile caps shall comply with ACI 117.



#### 7.4.2.7 Finishes.

7.4.2.7.1 Formed surfaces. Exposed surfaces of the foundation shall comply with Sec. 6.4.7.1. Finishing is not required on other formed surfaces. Defects and tie holes shall be patched.

7.4.2.7.2 Unformed surfaces. Unless otherwise specified, unformed surfaces shall have a floated finish.

#### 7.4.3 Piles.

7.4.3.1 Installation procedure. A procedure for installing piles shall be developed based on an engineering evaluation of available information, including geotechnical investigations, load tests, or other data.

7.4.3.2 Records. The installation and construction of piles shall be documented to confirm that pile installation criteria have been met. Records shall include information on pile testing, equipment, and installation operations.

7.4.3.3 Concrete. Cast-in-place concrete for piles shall comply with Sec. 7.4.2.1 through Sec. 7.4.2.3. Minimum specified compressive strength  $f'_c$  for piles shall be 3,000 psi (21 MPa).

7.4.3.4 Welding. Welding of steel piles and accessories shall comply with AWS D1.1. Welding of reinforcement shall comply with AWS D1.4.

7.4.3.5 Pile tolerances. Unless otherwise specified, tolerances for piles shall comply with the following:

- |                                      |                                |
|--------------------------------------|--------------------------------|
| 1. Variation from axial alignment:   | 2 percent of pile length       |
| 2. Variation from plan location:     | 3 in. (76 mm)                  |
| 3. Variation from cut-off elevation: | +1 in. (25 mm), -3 in. (76 mm) |

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## SECTION 8: ACCESSORIES

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### Sec. 8.1 General

8.1.1 *Scope.* This section covers materials, details, and design of accessories required for operation and maintenance and optional accessories commonly specified.

8.1.2 *Design.* Design loads shall not be less than required by ASCE 7. Design of safety accessories shall comply with OSHA 29 CFR 1910.

8.1.3 *Attachment to structure.* Attachments to the steel tank shall be connected by welding. Attachments to the concrete support structure shall be connected

by cast-in-place or post-installed anchors. Exterior anchorage devices shall be galvanized or stainless steel.

8.1.4 *Galvanic and chemical corrosion.* Dissimilar materials shall be isolated, as required, to prevent corrosion.

8.1.5 *Coating systems.* Unless otherwise specified, coating systems for steel accessories shall comply with the following:

1. Zinc (galvanized) coating shall be in accordance with ASTM A123.
2. Paint system for accessories within the steel tank or concrete support structure shall be in accordance with Sec. 5.4.5 (interior system).
3. Paint system for exterior accessories shall be in accordance with Sec. 5.4.5 (exterior system).

## **Sec. 8.2 Ladders**

8.2.1 *General.* Configuration, clearances, design, and details of ladders and related safety devices shall comply with OSHA 29 CFR 1910.

8.2.2 *Location.* Ladders shall be provided for access at the following locations:

1. Grade to upper platform.
2. Upper platform to tank floor manhole.
3. Upper platform to tank roof (mounted on access tube interior).
4. Tank roof to tank floor (mounted on access tube exterior) when specified.

8.2.3 *Fall protection.* When fall protection is required, provide safe climbing devices unless cages or other means of fall protection are specified. Caged ladders shall be offset at platforms.

## **Sec. 8.3 Platforms and Railings**

8.3.1 *Materials.* Unless otherwise specified, platforms and railings shall be painted or galvanized steel.

8.3.2 *Railing.* Railing shall have a nominal height of 42 in. (1,070 mm) and shall consist of top rail, intermediate rail, and posts.

8.3.3 *Toeboards.* Toeboards shall have a nominal vertical height of 4 in. (100 mm) with maximum ¼ in. (6 mm) clearance above platform level.

8.3.4 *Platforms.* Platforms shall have railings and toeboards at open sides, so arranged as to give safe access to the ladder.

8.3.4.1 *Upper platform.* Provide a minimum 48-in. (1,220-mm) wide platform below the tank floor for access from the wall ladder to the access tube interior ladder, tank floor manhole, and exterior rigging rail access opening.

8.3.4.2 *Intermediate platforms.* Provide minimum 36-in. (910-mm) wide by 36-in. (910-mm) long intermediate platforms when specified or where offset ladders are used.

8.3.5 *Roof railing.* Provide a railing to encircle centrally located roof hatches, vents, and other roof equipment when specified.

## Sec. 8.4 Access Openings

8.4.1 *Support wall doors.* Provide a personnel door. When specified, provide vehicle doors or other doors. Unless otherwise specified, doors shall comply with the following requirements.

8.4.1.1 *Personnel doors.* Door size shall be 36 in. × 84 in. × 1¾ in. (910 mm × 2,130 mm × 45 mm). Door and frame shall be minimum 18-gauge (1.21-mm) hollow metal construction designed for exterior use. Hardware shall include stainless-steel hinges, heavy-duty door closer, and lockset.

8.4.1.2 *Vehicle doors.* Door shall be rolling steel type, mounted on interior face of wall, with manual chain hoist operator. The curtain shall be minimum 22-gauge (0.85-mm) galvanized steel designed for 20-psf (960-Pa) wind load. Steel pipe bollards shall be provided at sides of openings for impact protection.

8.4.2 *Exterior rigging rail access.* Provide a minimum 24-in. (610-mm) opening at the top of the support wall for access to the exterior rigging rail from the upper platform. The opening shall be provided with a hinged or removable cover that may be screened and louvered to satisfy all or part of the support structure ventilation requirements.

8.4.3 *Access tube.* Provide a minimum 42-in. (1,070-mm) diameter steel access tube through the steel tank for access from the upper platform to the tank roof. Provide a minimum 30-in. (760-mm) roof opening equipped with a hinged weatherproof cover having inside and outside handles and interior locking device. Hatch cover shall be painted or galvanized steel, stainless steel, or aluminum. The force required to open the cover shall not exceed 30 lb (133 N).

8.4.4 *Tank roof openings.* Unless otherwise specified, provide at least two 30-in. (760-mm) minimum roof openings for access to ladders and rigging rails in the tank. One opening shall be a roof hatch with a 4-in. (100-mm) curb, weatherproof cover, and locking device. Roof hatch covers shall be painted or galvanized steel, stainless steel, or aluminum.

8.4.5 *Tank floor manhole.* Provide a watertight manhole in the tank floor that is accessible from the upper platform. The opening shall be 30 in. (760 mm)

or larger. Manholes shall be painted or galvanized steel or stainless steel. Other materials may be used for parts of the manhole not in contact with water.

### **Sec. 8.5 Permanent Rigging Devices**

Unless otherwise specified, provide the following:

1. An exterior continuous rail near the top of the concrete support structure. The rail may be attached to the support wall or steel tank.
2. Rails or other rigging devices that provide complete access for painting the tank interior.
3. Interior rigging devices near the top of the support wall for access to or maintenance of piping and equipment.

### **Sec. 8.6 Ventilation**

**8.6.1 Support structure ventilation.** Unless otherwise specified, provide one vent with a minimum net free vent area of 500 in.<sup>2</sup> (0.322 m<sup>2</sup>). The vent shall be located near the top of the support structure and shall be accessible from an interior ladder, platform, or floor. Vents shall be constructed of corrosion-resistant and durable materials, and shall be equipped with louvers and insect screens.

#### **8.6.2 Tank ventilation.**

**8.6.2.1 General.** Provide a roof vent and a pressure-vacuum relief mechanism or a single unit combining both functions. Vent and pressure-vacuum relief mechanism each shall have sufficient capacity to prevent excessive differential pressure between the atmosphere and tank interior at maximum flow rates.

**8.6.2.2 Vent.** The vent body shall be constructed of corrosion-resistant and durable materials with an overhanging cap having minimum 4-in. (100-mm) clearance above the roof surface. The vent shall attach to a flanged opening in the tank roof. Unless otherwise specified, vents shall be provided with aluminum, brass, or fiberglass insect screens.

**8.6.2.3 Pressure-vacuum relief mechanism.** The pressure-vacuum relief mechanism shall be constructed of corrosion-resistant and durable materials and shall attach to a flanged opening in the tank roof. It shall be designed to operate in the event of vent failure and return to normal position after relieving the pressure differential.

### **Sec. 8.7 Piping**

**8.7.1 General.** Provide inlet/outlet and overflow riser piping. Steel pipe shall be used within the steel tank. Unless otherwise specified, inlet/outlet and overflow piping located outside the stored water area shall be steel or stainless steel.

**8.7.2 Steel piping requirements.** Unless otherwise specified, steel piping shall comply with the following:

1. Pipe and fittings shall comply with ANSI/AWWA C200.
2. Connections may be flanged or welded. Flanged connections are required on pipe with a lining that is not accessible for repair after welding.
3. Wall thickness shall be based on a minimum internal pressure of 150 psi (1.03 MPa) and an allowable hoop tension stress of  $0.44F_y$ , but not less than  $\frac{3}{8}$  in. (9.52 mm). Overflow piping located outside the stored water area may be  $\frac{1}{4}$ -in. (6.35-mm) minimum.
4. Lining of the inlet/outlet or overflow piping, when specified, shall comply with ANSI/AWWA C205, ANSI/AWWA C210, or ANSI/AWWA C213.
5. Exterior coating system shall be as specified for the interior of the steel tank.

**8.7.3 Stainless-steel piping requirements.** Unless otherwise specified, stainless-steel piping shall comply with the following:

1. Pipe and fittings shall comply with ANSI/AWWA C220. Follower flanges used with stainless-steel rolled angle face rings shall be galvanized or painted steel.
2. Connections may be flanged or welded.
3. Wall thickness shall be based on a minimum internal pressure of 150 psi (1.03 MPa) and an allowable hoop tension stress of  $0.44F_y$ . Pipe 24-in. (610-mm) diameter and smaller shall have a diameter-to-thickness ratio of 130 or less, and minimum thickness of 14 gauge (1.98 mm). Minimum thickness of larger pipe is  $\frac{1}{4}$  in. (6.35 mm).

**8.7.4 Inlet/outlet piping.** Provide inlet/outlet piping that complies with the following:

1. Unless otherwise specified, provide a single inlet/outlet pipe. Diameter shall be as specified.
2. Entrance shall be at the low point of the tank, unless a separate drain is provided. Provide entrance safety protection for pipe larger than 12 in. (305 mm). When specified, provide a removable silt stop below the BCL that projects minimum 6 in. (150 mm) above the steel liner.
3. For steel pipe 36 in. (910 mm) or larger, provide an access manhole near grade.
4. Provide support brackets, hangers, and guides at intervals not exceeding 20 ft (6.1 m). Supports shall be designed for static, dynamic, and thermal loads.



5. Provide an expansion joint to accommodate differential movement. It shall be accessible for inspection and maintenance.

**8.7.5 Overflow piping.** Provide an overflow pipe that complies with the following:

1. Overflow pipe shall be sized for the specified maximum fill rate. Minimum overflow pipe diameter shall be 6 in. (150 mm), unless a larger size is specified.

2. Overflow pipe within the steel tank may be attached to the interior or exterior of the access tube, unless the location is specified. Horizontal piping below the tank floor shall be sloped for drainage.

3. Overflow pipe entrance shall have a vortex prevention device and an entrance weir if required. Overflow inlet capacity shall not be less than the specified maximum fill rate, based on water level cresting 6 in. (150 mm) above the overflow lip.

4. Provide support brackets, hangers, and guides at intervals not exceeding 20 ft (6.1 m). Supports shall be designed for static, dynamic, and thermal loads.

5. Unless otherwise specified, overflow pipe shall discharge at grade onto a concrete splash pad that drains water away from the structure. A corrosion-resistant coarse mesh screen or a flap valve shall be attached to the end of the overflow pipe.

**8.7.6 Connecting piping.** Piping (pipe, valves, and fittings) that connects to riser piping shall be specified. Connecting piping shall have sufficient flexibility to accommodate differential movement.

**8.7.7 Pipe cover.** Unless otherwise specified, minimum pipe cover shall be determined from Figure 2. Pipe cover less than shown in Figure 2 may be provided when the pipe is sufficiently insulated.

## **Sec. 8.8 Lightning Protection**

Provide a lightning protection system in accordance with NFPA 780. Minimum requirements include two 28-strand by 14-gauge copper down conductors 180 degrees apart. Down conductors shall be bonded to the steel tank and fastened to the interior of the support wall at 3-ft (0.91-m) maximum spacing. The down conductors shall connect to ground terminals appropriate for the site soil conditions.

## **Sec. 8.9 Electrical and Lighting**

**8.9.1 General.** Electrical equipment and wiring shall comply with NFPA 70. Installations within the support structure and access tube shall comply with

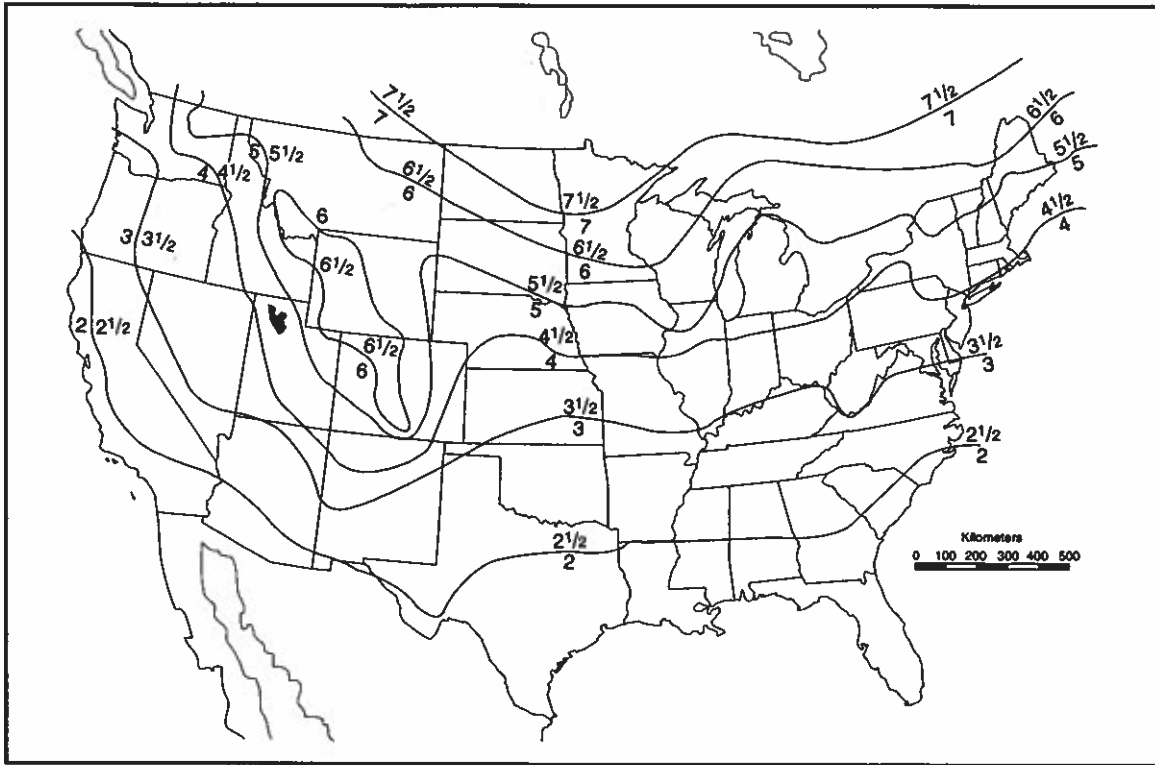


Figure 2 Minimum pipe cover (cover above top of pipe in feet) by geographic area

damp location requirements, and exterior installations shall comply with wet location requirements.

8.9.2 *Electrical equipment.* Unless otherwise specified, provide the following:

8.9.2.1 Interior lights. Ladder, platform, and access tube fixtures shall be 150-W incandescent type with aluminum body, clear glass globe, and guard. Fixtures shall be provided at the following locations:

1. Along the support wall 8 ft (2.4 m) above the slab-on-grade at 30-ft (9.1-m) maximum horizontal spacing.
2. Adjacent to access ladders at 25-ft (7.6-m) maximum vertical spacing. Locate the lower fixture 8 ft (2.4 m) above the slab-on-grade.
3. Above each platform.
4. At the top and bottom of the interior of the access tube.

8.9.2.2 Exterior lights. Fixtures shall be 100-W metal halide with aluminum base housing, low temperature ballast, and photocell control. Provide a fixture above each personnel door and vehicle door.



8.9.2.3 Receptacles. Standard convenience outlets shall be heavy-duty, corrosion-resistant, three-wire duplex receptacles. Provide outlets adjacent to the power distribution panel, at the upper platform, and in the top of the access tube. In-line ground fault circuit interrupters shall be provided at each outlet. Each outlet shall consist of a two-gang cast metal box with receptacle, in-line ground fault circuit interrupter, and damp location covers.

8.9.2.4 Obstruction lighting. Provide obstruction lighting in accordance with Federal Aviation Administration (FAA) standards when specified.

8.9.3 *Wiring materials.* Unless otherwise specified, wiring materials shall comply with the following:

8.9.3.1 Conduit. Exposed interior conduit shall be galvanized electrical metallic tubing. Exposed exterior conduit shall be galvanized intermediate or rigid metal conduit. Underground conduit shall be rigid nonmetallic PVC conduit, schedule 40. Exposed sleeves embedded in the concrete wall shall be stainless steel or PVC. Conduit sealing fittings shall be installed between sleeves and conduit. Conduit shall be installed such that it does not obstruct ladder access, create a tripping hazard, or create a headroom obstruction.

8.9.3.2 Fittings and boxes. Boxes, fittings, and device plates shall be galvanized or aluminum. The material of the fitting or device shall match the conduit type. Exterior applications shall be waterproof.

8.9.3.3 Enclosures. Load centers, power distribution panels, lighting panels, and enclosed switches shall be in NEMA 12 enclosures for interior locations and NEMA 3R for exterior locations, as defined in NEMA publication NEMA 250.

8.9.3.4 Grounding. Grounding shall be provided in accordance with NFPA 70.

## Sec. 8.10 Interior Floors

8.10.1 *General.* Concrete materials and construction shall comply with ACI 318. Tolerances for concrete shall comply with ACI 117. Fabrication and erection of structural steel shall comply with AISC Code of Standard Practice for Steel Buildings and Bridges.

8.10.2 *Slab-on-grade.* Unless otherwise specified, provide a concrete slab-on-grade within the support structure in accordance with the following:

1. Concrete materials and construction shall comply with ACI 318. Tolerances for concrete shall comply with ACI 117.

2. Minimum 5-in. (125-mm) thickness, and minimum concrete compressive strength  $f'_c$  of 3,500 psi (24 MPa).

3. Minimum reinforcement ratio  $\rho_g$  of 0.0018. Reinforcement shall be placed 2 to 3 in. (51 mm to 76 mm) below the top surface of the slab at a maximum spacing of 18 in. (460 mm).

4. Provide a troweled finish. Slope the top surface to floor drains or to doorways if drains are not specified.

5. Provide full depth isolation joints at junctions with walls, columns, foundations, and other points of restraint.

6. Provide 1¼-in. (32-mm) depth contraction joints at 30-ft (6.1-m) maximum spacing.

7. Provide suitable subgrade. Fill materials shall be compacted to 95 percent standard Proctor density (ASTM D698).

8.10.3 *Intermediate floors.* The following requirements apply when intermediate floors are specified:

1. Concrete materials, design, and construction shall comply with ACI 318. Tolerances for concrete shall comply with ACI 117.

2. Fabrication and erection of structural steel shall comply with ANSI/AISC Code of Standard Practice for Steel Buildings and Bridges. Structural steel design shall comply with ANSI/AISC 360.

3. Unless otherwise specified, minimum uniform live load shall be 100 psf (4.8 kPa). The safe floor live load shall be posted at each floor level.

4. Unless otherwise specified, provide a troweled finish.

## **Sec. 8.11 Antennas and Communication Equipment**

Antennas and communication equipment shall comply with the following:

1. Loads from antennas and communication equipment shall be specified, and attachments to the structure shall be designed to prevent overstress in the support elements.

2. Cables, conduit, or other components shall not be attached to ladders. Cables shall be routed through conduit, cable trays, or other separate supports. Provide support per NFPA 70 and applicable manufacturer's product data sheets.

3. Roof penetrations shall be made with weatherproof fittings designed for the conduit or antenna cable.

4. Installation of equipment and access to the tank and tank accessories shall comply with OSHA.

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## SECTION 9: INSPECTION AND TESTING

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### Sec. 9.1 General

9.1.1 *Scope.* This section covers inspection and testing requirements for the foundation, concrete support structure, steel tank, and accessories.

9.1.2 *Notation.* The following symbols are used to represent variables in this section:

$E$  = weld size defined in Sec. 9.5.4.2, in. (mm)

$L$  = largest dimension of a radiographed discontinuity, in. (mm)

9.1.3 *Testing agencies—concrete.* Agencies that perform testing services on concrete materials shall meet the requirements of ASTM C1077. Testing agencies that perform testing services on reinforcing steel shall meet the requirements of ASTM E329.

9.1.4 *Inspection and testing personnel—welding.* Inspection and testing of welded joints shall only be performed by qualified personnel.

9.1.4.1 *Welding inspectors.* Inspectors responsible for acceptance or rejection of materials and workmanship shall meet one of the following requirements:

1. Current or previous certification as an AWS certified welding inspector (CWI) in accordance with the provisions of AWS QC1, Standard and Guide for Qualification and Certification of Welding Inspectors.

2. Current or previous qualification by the Canadian Welding Bureau (CWB) to the requirements of Canadian Standard Association (CSA) Standard W178.2, Certification of Welding Inspectors.

3. An engineer or technician who meets other minimum certification requirements specified by the purchaser, or if none are specified by the purchaser, a licensed engineer or technician, trained in metals fabrication, inspection, and testing, who is competent to perform inspection of the work.

9.1.4.2 *NDT personnel.* Personnel performing nondestructive testing (NDT) of weld joints shall be qualified in accordance with the current edition of the American Society for Nondestructive Testing, Recommended Practice No. SNT-TC-1A. Evaluation for acceptance of radiographic tests shall only be performed by individuals qualified for NDT Level II or NDT Level III.

9.1.5 *Test and inspection reports.* The results of tests and inspections performed during the course of the work shall be reported. When specified or required

**Table 13a Specific reporting requirements**

Section	Documentation
9.2.2 (item 3)	Report of installation and construction of piles per Sec. 7.4.3.2.
9.4.1	Manufacturer's report of steel material, when specified.
9.4.2.2	Report of field measurements for compliance with tolerances for designated shell plates.
9.5.1.2	Report and certification of weld inspection, when specified.
9.5.4.5	Record of location of radiographs.
9.7.2	Settlement monitoring, when specified.

**Table 13b Other reports of tests and inspections, when specified**

Section	Documentation
9.1.3	Conformance to ASTM standards of concrete testing agencies.
9.1.4	Certification of welding inspectors.
9.2.1	Report of inspection of foundation-bearing stratum.
9.2.2	Report of inspection of deep foundation construction.
9.2.3	Minimum compaction requirements for backfill.
9.3.1	Verification of compliance for concrete materials and mixtures.
9.3.2	Inspection of concrete reinforcement and embedments.
9.3.3	Report of concrete testing during construction.
9.3.4	Report of tests on hardened concrete.
9.4.2.1	Report of visual inspection of steel tank for compliance with tolerances.
9.5.1	Report of visual inspection of steel tank welding for compliance with Sec. 5.4.
9.6.1	Report of inspection of attachments to concrete support structure.
9.6.2	Report of inspection and testing of operating equipment.
9.7.1	Report of leak testing of the tank.
9.7.2	Report of settlement monitoring.

by this standard, furnish reports and certifications in accordance with sections listed in Tables 13a and 13b.

## Sec. 9.2 Geotechnical and Foundation

9.2.1 *Shallow foundations.* The exposed bearing stratum of shallow foundations shall be inspected to ensure that the material encountered reflects the findings and recommendations of the geotechnical investigation.

9.2.2 *Deep foundations.* Field inspection of pile installation and concrete work shall comply with the following:

1. Continuous inspection shall be made during pile-driving operations.
  2. Periodic inspection shall be made during construction of drilled piers or auger-cast piles, during placement of concrete, and on completion of placement of reinforcement.
  3. Documentation of pile installation shall comply with Sec. 7.4.3.2.
- 9.2.3 *Soil backfill.* Inspect and test backfill soils for conformance to requirements of Sec. 7.4.1.2.

### Sec. 9.3 Concrete

9.3.1 *Materials.* Verify the materials and concrete mixtures comply with this standard.

9.3.2 *Inspection.* Inspect reinforcement and embedded items prior to placing concrete. Tolerances shall comply with Sec. 6.4.6, Sec. 7.4.2.6, and Sec. 7.4.3.5.

9.3.3 *Concrete testing.* Perform concrete testing during construction in accordance with Sec. 9.3.3.1 through Sec. 9.3.3.6.

9.3.3.1 *Sampling.* Obtain composite samples in accordance with ASTM C172. Each sample shall be obtained from a different batch of concrete on a random basis. Obtain at least one composite sample for each 100 yd<sup>3</sup> (76 m<sup>3</sup>), or fraction thereof, for each design mixture placed in any given day.

9.3.3.2 *Strength tests: laboratory-cured specimens.* Perform strength tests for each composite sample in accordance with the following procedure:

1. Mold four cylinders from each sample and standard cure in accordance with ASTM C31. Record any deviations from ASTM requirements in the test report.

2. Test cylinders in accordance with ASTM C39. Test one specimen at 7 days for information, and two specimens at 28 days for acceptance. If one specimen in a test shows evidence of improper sampling, molding, or testing, discard the specimen and test the remaining cylinder. The compressive strength test results for acceptance shall be the average of the compressive strengths of the two acceptable specimens tested at 28 days. If one specimen in a test shows evidence of improper sampling, molding, or testing, discard the specimen and consider the remaining cylinder to be the test result. If both specimens in a test show any defects, discard the entire test.

9.3.3.3 *Strength tests: field-cured specimens.* Perform strength tests on field-cured cylinders in accordance with the following where knowledge of early age strength of concrete is required for formwork removal or construction loading:

1. Mold at least one cylinder from each sample in accordance with ASTM C31, and cure under the same conditions for moisture and temperature as used for the concrete the samples represent.

2. Test cylinders in accordance with ASTM C39.

9.3.3.4 Slump. Determine slump in accordance with ASTM C143 for each strength test sample and whenever consistency of concrete appears to vary.

9.3.3.5 Temperature. Determine temperature of each strength test sample, using ASTM C1064.

9.3.3.6 Air content. Determine air content of each test sample, using ASTM C231, ASTM C173, or ASTM C138. Perform additional tests as necessary for control.

9.3.4 *Tests on hardened concrete.* Tests on hardened concrete shall be performed when required to investigate low-strength concrete.

9.3.4.1 Nondestructive tests. The rebound hammer in accordance with ASTM C805, pulse velocity methods in accordance with ASTM C597, or other methods may be used to evaluate the uniformity and relative concrete strength in place, or for selecting areas to be cored.

9.3.4.2 Core tests. Cores shall be obtained and tested in accordance with ASTM C42. At least three representative cores shall be taken from each member or area of concrete in place that is considered potentially deficient. The location of cores shall be selected to impair the strength of the structure as little as possible. If, before testing, the cores show evidence of having been damaged subsequent to removal from the structure, replacement cores shall be taken. Fill core holes with low slump concrete or mortar having a strength equal to or greater than the original concrete.

9.3.5 *Acceptance of concrete strength.*

9.3.5.1 Evaluation. Evaluation of standard molded and cured strength specimens, nondestructive tests, and core tests shall be valid only if tests have been conducted in accordance with specified procedures.

9.3.5.2 Standard molded and cured strength specimens. The strength level of concrete will be considered satisfactory when the averages of all sets of three consecutive compressive strength tests equal or exceed the specified compressive strength and no individual strength test result falls below the specified compressive strength by more than 500 psi (3.45 MPa).

9.3.5.3 Core tests. Strength level of concrete in the area represented by core tests will be adequate when the average compressive strength of the cores is

equal to at least 85 percent of the specified compressive strength and if no single core is less than 75 percent of the specified compressive strength.

#### **Sec. 9.4 Steel Tank: Materials and Tolerances**

9.4.1 *Materials.* Verify that materials for the steel tank comply with the standard. A report from the original manufacturer shall be provided for all plate and structural material when specified. The documentation shall include certification of test results and other information required by ASTM A6 or ASTM A20.

##### 9.4.2 *Tolerances.*

9.4.2.1 *Visual inspection.* Perform visual inspection supplemented by measurements to verify that the steel tank complies with the tolerances in Sec. 5.4.4.

9.4.2.2 *Special inspection for local imperfection.* In accordance with Sec. 5.3.5.3 (item 6), inspect each shell plate designated on construction drawings or the Quality Assurance (QA) Plan as follows:

1. As a minimum, measure the meridional profile at each weld seam and midway between each meridional weld seam.
2. Perform additional measurements at locations where the profile appears irregular or where specified tolerances of Sec. 5.4.4.2 are exceeded. Determine the extent of any out-of-tolerance areas.
3. Document field measurements, and certify compliance or noncompliance with tolerance requirements of Sec. 5.4.4.2.

#### **Sec. 9.5 Steel Tank: Welding**

9.5.1 *General.* Perform welded joint inspection during fabrication of components and construction of the steel tank in accordance with Sec. 9.5.2.

9.5.1.1 *Time of inspection.* Perform required inspection of welds as the work progresses. Where access is from movable staging, perform inspection as soon as practicable after completion of welding at any staging position.

9.5.1.2 *Reporting.* When specified, a report shall be prepared certifying that the work was inspected in accordance with the requirements of this section. The report shall include the following:

1. A copy of welder performance qualifications specified in Sec. 5.4.1.2.
2. A summary of inspection of radiographs and other inspections, including location of tests on developed shell plate diagrams.
3. Identification of defective welds and a statement of action taken to repair.
4. Record of welders employed on each joint.

### 9.5.2 *Inspection requirements.*

9.5.2.1 *Visual inspection.* Perform visual inspection of all welds in accordance with Sec. 9.5.3. Visual inspection shall be performed prior to conducting other tests.

9.5.2.2 *Shell plate joints.* Inspect by radiographic examination in accordance with Sec. 9.5.4 complete joint penetration welds in shell plate specified in Sec. 5.3.3.5 (item 1). Complete joint penetration shell plate welds that cannot be radiographed may be inspected by air carbon arc gouging in accordance with Sec. 9.5.5. For inspection purposes, primary stress joints subject to compression stress shall be considered secondary stress joints. The number and location of radiographs shall comply with the following:

1. *Primary stress joints.* Primary stress joints of the same type and thickness shall have one radiograph taken in the first 10 ft (3 m) of completed joint welded by each welder. One additional radiograph shall be taken in each additional 100 ft (30 m) of welded joint or any fraction exceeding 50 ft (15 m), without regard to the number of welders. The radiograph locations selected for primary stress joints shall include 5 percent of all junctures of joints that include at least one primary stress joint, with a minimum of two such junctures per tank.

2. *Secondary stress joints.* Secondary stress joints of the same type and thickness shall have one radiograph taken in the first 10 ft (3 m) of completed joint and one additional radiograph in each additional 200 ft (60 m) or any fraction exceeding 100 ft (30 m), without regard to the number of welders.

3. *Plates of same thickness.* Plates shall be considered the same thickness for the purpose of establishing radiograph requirements if the difference in the specified plate thickness does not exceed  $\frac{1}{8}$  in. (3.18 mm).

4. *Multiple welders.* If opposite sides of a joint are completed by different welders, it is permissible to test both welders' work with one radiograph.

9.5.2.3 *Liner plate joints.* Liner plate joints in contact with water shall be tested by the pressure/vacuum method described in Sec. 9.5.6.

9.5.3 *Visual inspection.* Visually inspect all welds for compliance with Sec. 5.4. Welds will be considered satisfactory if the following acceptance criteria are met:

1. No evidence of cracks, regardless of size or location.
2. Thorough fusion exists between adjacent layers of weld metal and between weld metal and base metal.
3. All craters are filled to provide specified weld size.



## 4. Porosity:

- a. None in any butt joints subject to primary stress.
  - b. For all other groove welds, the sum of visible porosity greater than  $\frac{1}{32}$  in. (0.80 mm) in diameter shall not exceed  $\frac{3}{8}$  in. in any linear inch of weld (9 mm in any 25-mm length), and shall not exceed  $\frac{3}{4}$  in. in any 12 in. of weld (19 mm in any 305-mm length).
5. Acceptable weld profiles that comply with Sec. 5.4.4.2 (item 3).
  6. Alignment of butt joints within tolerance limits of Sec. 5.4.4.2 (item 2).

## 9.5.4 Radiographic examination.

9.5.4.1 Radiographic examination method. Except as modified in this section, radiographic procedure and technique shall be in accordance with ASME Boiler and Pressure Vessel Code Section V, Nondestructive Examination, Article 2.

1. Surface preparation. The finished surface of the weld joint at the location of the radiograph shall be flush with the plate or have weld reinforcement not exceeding the limits of Table 14. Weld ripples or surface irregularities shall be removed to the degree that they do not mask or can be confused for any discontinuity in the resulting radiographic image.

2. Film. Film shall be of sufficient size to provide at least  $\frac{1}{2}$  in. (12.7 mm) of film beyond the projected edge of the weld and have sufficient space for placement of identification markers and image quality indicators. Radiographs shall clearly show a minimum of 6 in. (150 mm) of weld length along longitudinal and circumferential joints. Radiographs at junctions of circumferential welds and longitudinal welds shall clearly show 2 in. (51 mm) of circumferential weld length on each side of the intersecting joint, and a minimum of 3 in. (76 mm) weld length on the longitudinal seam.

3. Image quality indicators (IQIs). Image quality indicator selection shall be in accordance with ASME Boiler and Pressure Vessel Code, Section V, Article 2, T-276, based on the average thickness of the two plates joined, plus weld reinforcement. The use of IQIs shall be in accordance with ASME Boiler and Pressure Vessel

**Table 14 Maximum reinforcement of shell butt joints at radiograph locations**

Shell Plate Thickness, $t$		Maximum Height of Weld Reinforcement	
(in.)	(mm)	(in.)	(mm)
$t \leq \frac{1}{2}$	$t \leq 12.7$	$\frac{1}{16}$	1.59
$\frac{1}{2} < t \leq 1$	$12.7 < t \leq 25$	$\frac{3}{32}$	2.38
$t > 1$	$t > 25$	$\frac{1}{8}$	3.18

Code, Section V, Article 2, T-277. At least one IQI shall be used for each film located near the center of the location to be examined. Wire-type IQIs shall be placed across the weld, and hole-type IQIs shall be placed parallel to the weld seam.

4. Evaluation. Evaluation of radiographs as to suitability for detecting weld defects shall be in accordance with ASME Boiler and Pressure Vessel Code, Section V, Article 2, T-280. Acceptance of radiographs shall be based on the ability to display the prescribed hole-type IQI image and 2T hole, or the essential wire of a wire-type IQI.

9.5.4.2 Definitions for radiographic evaluation. The following definitions are applicable to evaluation of radiographic tests.

- *Aligned discontinuity (elongated)*: A sequence of elongated discontinuities where the major axes of each discontinuity are approximately aligned.
- *Discontinuity*: An interruption of the typical structure of a weldment, such as a lack of homogeneity in the mechanical, metallurgical, or physical characteristics of the material weldment. A discontinuity is not necessarily a defect.
- *Elongated discontinuity*: A discontinuity where its length exceeds three times its width.
- *Rounded discontinuity*: A discontinuity where its length is three times its width or less, where it may be round or irregular, and where it may have tails.
- *Weld size*: Thickness of the weld exclusive of any reinforcement. For butt joints connecting plates of different thickness, it is the thinner of the two thicknesses.

9.5.4.3 Acceptance criteria. Radiographs shall be examined for cracks and discontinuities by radiographers meeting the qualification requirements of Sec. 9.1.4.2.

Weld joints subject to radiographic testing in addition to visual inspection that have cracks or any of the following defects shall be repaired in accordance with Sec. 9.5.4.4 and Sec. 9.5.7.

1. Any incomplete fusion or penetration in primary stress joints. For secondary stress joints, incomplete fusion or penetration shall be evaluated on the basis of being either elongated or rounded discontinuities.

2. Individual elongated discontinuities of length  $L$  greater than  $\frac{1}{4}$  in. (6.35 mm) the length of which  $L$  exceeds the smaller of  $\frac{3}{4}$  in. (19 mm) or  $\frac{2}{3} E$ .

3. Elongated aligned discontinuities where the sum of greatest dimensions exceeds  $E$  in a length of  $6E$ , except when the clearance between every pair of adjacent discontinuities is greater than  $3L$ , where  $L$  is the largest dimension of the discontinuities being considered. When the length of weld being examined is less than  $6E$ , the sum of greatest dimensions shall be proportionately reduced.

4. Clearance of individual discontinuities to another discontinuity or to an edge or end of an intersecting weld less than  $3L$ , where  $L$  is the largest dimension of the discontinuities being considered.

5. Rounded indications in excess of those shown as acceptable in ASME Boiler and Pressure Vessel Code, Section VIII, Division I, appendix 4.

9.5.4.4 Defective welds. Where a weld defect is located within 3 in. (76 mm) of the edge of a film, an additional radiograph on the defect side shall be taken to determine the extent of unacceptable weld. If the additional radiograph fails to meet the criteria of Sec. 9.5.4.3, the extent of unacceptable weld shall be determined by additional radiographs or by air carbon arc gouging. A radiograph shall be taken at the end of air carbon arc gouging to verify the entire defect has been removed. Alternatively, all welding performed by the welder on the joint shall be replaced and an additional radiograph taken of any other joint completed by the same welder. Acceptance criteria for additional radiographs shall comply with Sec. 9.5.4.3.

9.5.4.5 Record of radiographic inspection. A record of radiographic inspection shall be made of all films with their identification marks on a developed shell plate diagram.

9.5.5 *Inspection by air carbon arc gouging.* For each location, approximately 2-in. length shall be gouged out to the root of the weld. Visual inspection shall be made for lack of penetration or fusion, cracks, or porosity. If the weld is considered defective, then additional adjacent areas shall be gouged to define the limit of unacceptable weld. All gouged areas shall be repair-welded using an accepted procedure.

A record shall be made of any inspection by air carbon arc gouging. The joint shall be identified, results of inspection documented, and reason noted for utilizing this type of inspection.

9.5.6 *Pressure/vacuum testing.* Joints shall be covered with linseed oil, soap solution, or other suitable material for the detection of leaks. A minimum 2-psi (0.0138-MPa) partial vacuum below atmospheric pressure shall be applied to the joint for vacuum testing.

9.5.7 *Repair of defective welds.* Defective welds shall be removed by grinding, chipping, or by air arc or oxygen gouging from one or both sides of the joint. The joint shall be rewelded in compliance with the approved procedures. Removal of defective welds is required only to the extent necessary to remove the defects encountered. Repairs shall be reinspected by the original test procedure.

**Sec. 9.6 Accessories**

9.6.1 *Attachments.* Inspect for compliance with drawings and specification the attachment of ladders, platforms, brackets, and equipment to the support structure.

9.6.2 *Equipment.* Inspect all operating equipment to ensure it is properly installed, tested, and in working order.

**Sec. 9.7 Hydrotest**

9.7.1 *Leak test.* Unless otherwise specified, test the tank by filling with water to the TCL, and inspect for leakage at weld seams. Repair any leaks in accordance with Sec. 9.5.7 after lowering the water level at least 2 ft (0.61 m) below the point of repair.

9.7.2 *Settlement monitoring.* When specified, monitor the foundation for settlement during the hydrotest using surveying methods in accordance with the following:

1. Install a minimum of four equally spaced permanent survey points to the support structure.
2. Prior to filling the tank, determine the elevation of survey points using a permanent or temporary benchmark outside the area that may be influenced by foundation. Record elevation of survey points during filling and on completion of filling.
3. Document settlement monitoring by reporting survey point locations, their initial elevations, and elevations or settlement during filling along with the water level at time of survey.

## APPENDIX A

### Commentary on Selected Sections\* of the Standard

*This appendix is for information only and is not part of ANSI/AWWA D107.*

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## SECTION A.1 GENERAL

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### Sec. A.1.1 Scope

This standard addresses composite elevated tanks that are used in water supply service and that use a welded steel tank for watertight containment and a single concrete support structure. This standard may be used for other applications (e.g., surge or process tanks) and structural configurations, provided the suitability of these requirements to such applications or configurations is determined.

### Sec. A.1.3 Application

See Sec. III.B of the foreword for using this standard in a project specification.

### Sec. A.1.4 Drawings, Calculations, and Instructions

Inclusion of the design basis on the drawings provides a way of capturing critical information that otherwise would only be found in design calculations.

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## SECTION A.2: REFERENCES

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The standard and commentary refer to specific sections, tables, or figures in ACI 318 and ASCE-7. These references are specific to

ACI 318-11—Building Code Requirements for Structural Concrete.

ASCE 7-10—Minimum Design Loads for Buildings and Other Structures.

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\* Numbers that appear with headings in this appendix reflect the original numbering of sections within the standard itself and commentary is provided only on selected sections.

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## SECTION A.4: GENERAL REQUIREMENTS FOR ANALYSIS AND DESIGN

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### Sec. A.4.2 Design Loads

A.4.2.1 *Minimum loads.* ASCE 7 minimum design loads are adapted to composite elevated tanks. The loads are for Risk Categories III and IV as defined in Table 1.5-1 of ASCE 7. The Risk Category IV classification includes structures designated as essential facilities required in emergencies, and is considered applicable to tanks used for potable water supply and fire protection. The Risk Category III classification is applicable to tanks used for purposes other than fire protection. Loads for both classifications are the same, except for a smaller importance factor associated with the Risk Category III classification. The structure is considered an essential facility when the stored water is required for fire protection or in an emergency.

The specifier should investigate and define in the project documents larger minimum design loads than those specified in this standard when required by applicable building codes, standards, or other specifications.

A.4.2.3 *Water load.* See Sec. A.4.3.4 for discussion of the basis of the load factor for water used for strength design in the standard.

A.4.2.4 *Live load.* Minimum roof snow load is 15 lb/ft<sup>2</sup> (720 Pa). ASCE 7 requires minimum roof live load of 12 lb/ft<sup>2</sup> (570 Pa). It is the intent of the standard that minimum roof snow load is adequate to account for the presence of workmen and materials during repair and maintenance. An appropriate roof live load, additive to the snow load, should be specified for those portions of the roof used for support of equipment or for other purposes.

Walkways and elevated platforms should be designed for minimum uniform distributed and concentrated live loads specified in ASCE 7.

A.4.2.5 *Snow load.* The provisions for snow load are a simplified and conservative application of ASCE 7, chapter 7, requirements. Only uniformly distributed snow loads are considered because significant unbalanced snow loads do not occur with the roof configurations typically furnished with composite elevated tanks.

Flat roof snow load is given by Eq 4-1a and Eq 4-1b. Minimum snow load given by Eq 4-1a is  $20I_s$ , rounded to 25 psf (1,200 Pa), and is for ground snow load greater than 20 psf (0.96 kPa). Minimum snow load given by Eq 4-1b is  $p_g I_s$  for

ground snow load of 20 psf (960 Pa) or less. The 15 psf (720 Pa) lower limit reflects the minimum live load considered appropriate for tank roofs. The 0.9 coefficient in Eq 4-1a is a combined factor that is the rounded product of the following ASCE 7 factors:

1. Flat roof factor: 0.7
2. Exposure factor,  $C_e$ : 0.9
3. Thermal factor,  $C_t$ : 1.2
4. Importance factor,  $I_f$ : 1.2

A.4.2.5.4 Sloped roof snow load. The equation for the roof slope factor  $C_r$  is based on Figure 7-2 of ASCE 7 with the reduction for slope beginning at 30 degrees. Roofs are considered free of snow when the roof slope angle  $\theta$ , exceeds 70 degrees.

#### A.4.2.6 Wind load.

A.4.2.6.1 Scope. The provisions for wind load are a simplified and conservative application of requirements in ASCE 7, chapters 26 and 29. The wind forces considered in this standard are for rigid structures. The potential for across-wind excitation or flutter should be investigated for tall slender tanks.

A.4.2.6.2 Basic wind speed. Factored basic wind speed  $V_b$  is 120 mph (54 m/sec) for all areas of the contiguous United States, except hurricane coastal areas and special wind regions.

A.4.2.6.3 Design wind force. Wind forces acting on the structure are positive and negative pressures acting concurrently on the projected area. The factored wind force determined by Eq 4-4 is applicable to a portion of the structure subjected to a constant wind pressure. 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. Design wind pressure  $C_f p_z$  used in Eq 4-4 corresponding to a basic wind speed  $V_b$  of 120 mph (54 m/sec) is tabulated in Tables A.1a and A.1b for convenience.

Equation 4-5 is used to determine wind force drag coefficient  $C_f$ . It is derived from ASCE 7, Figure 29.5-1, for moderately smooth, round cross sections with  $D\sqrt{q_x} > 2.5$  and having height-to-maximum-diameter ratios in the range of 1 to 7. The value of the drag coefficient  $C_f$  will vary between 0.5 and 0.6, and is comparable to the drag coefficient values that have historically been used by ANSI/AWWA D100 for cylindrical and doubly curved surfaces.

Only Exposure Categories C and D are considered applicable to elevated tanks. ASCE 7 defines these categories as follows:

- *Exposure C:* All cases where Exposure D does not apply.

**Table A.1a Factored design wind pressure  $C_f p_z$  (in psf) for  $V_b = 120$  mph\***

Height Zone (ft)	$C_f = 0.50$		$C_f = 0.55$		$C_f = 0.60$	
	Exposure C	Exposure D	Exposure C	Exposure D	Exposure C	Exposure D
0-25	24.0	24.0	26.4	26.4	28.8	28.8
25-50	24.0	24.0	26.4	26.4	28.8	28.8
50-75	24.0	25.1	26.4	27.6	28.8	30.1
75-100	24.0	26.4	26.4	29.1	28.8	31.7
100-125	24.4	27.5	26.9	30.2	29.3	32.9
125-150	25.4	28.3	27.9	31.2	30.5	34.0
150-175	26.2	29.1	28.9	32.0	31.5	34.9
175-200	27.0	29.8	29.7	32.8	32.4	35.8
200-225	27.7	30.4	30.4	33.5	33.2	36.5
225-250	28.3	31.0	31.1	34.1	33.9	37.2
250-275	28.9	31.5	31.8	34.6	34.6	37.8
275-300	29.4	32.0	32.3	35.2	35.3	38.4

\* Tabulated for gust factor  $G_f = 1.0$ . Where structure period  $T < 1.0$  second, tabulated values may be multiplied by 0.85. For basic wind speeds greater than 120 mph, the design wind pressure  $C_f p_z$  can be determined by multiplying the values from Table A.1a by  $(V_b/120)^2$ . Minimum factored design wind pressure  $C_f P_z = 48 C_f$ .

**Table A.1b Factored design wind pressure  $C_f p_z$  (in Pa) for  $V_b = 54$  m/sec\***

Height Zone (m)	$C_f = 0.50$		$C_f = 0.55$		$C_f = 0.60$	
	Exposure C	Exposure D	Exposure C	Exposure D	Exposure C	Exposure D
0-7.6	1,150	1,150	1,265	1,265	1,380	1,380
7.6-15.2	1,150	1,150	1,265	1,265	1,380	1,380
15.2-22.9	1,150	1,219	1,265	1,341	1,380	1,463
22.9-30.5	1,150	1,282	1,265	1,410	1,380	1,538
30.5-38.1	1,186	1,332	1,305	1,465	1,424	1,599
38.1-45.7	1,233	1,375	1,356	1,512	1,479	1,650
45.7-53.3	1,273	1,412	1,400	1,553	1,528	1,695
53.3-61.0	1,310	1,446	1,441	1,590	1,572	1,735
61.0-68.6	1,343	1,476	1,477	1,623	1,611	1,771
68.6-76.2	1,373	1,503	1,510	1,653	1,647	1,803
76.2-83.8	1,400	1,528	1,540	1,681	1,680	1,833
83.8-91.4	1,426	1,551	1,569	1,706	1,711	1,861

\* Tabulated for gust factor  $G_f = 1.0$ . Where structure period  $T < 1.0$  second, tabulated values may be multiplied by 0.85. For basic wind speeds greater than 54 m/sec, the design wind pressure  $C_f p_z$  can be determined by multiplying the values from Table A.1b by  $(V_b/54)^2/2,916$ . Minimum factored design wind pressure  $C_f P_z = 2,300 C_f$ .



- *Exposure D:* Exposure D is applicable where ground Surface Roughness D prevails in the upwind direction for a distance greater than 5,000 ft (1,520 m) or 20 times the structure height, whichever is greater. Exposure D shall extend into downwind areas of other ground surface roughness for a distance of 600 ft (180 m) or 20 times the structure height, whichever is greater.

Exposure categories are based on ground surface roughness determined from natural topography, vegetation, and constructed facilities, as follows:

- *Surface Roughness B:* Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.
- *Surface Roughness C:* Open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m). This surface roughness includes flat open country, grasslands, and all water surfaces in hurricane-prone regions.
- *Surface Roughness D:* Flat, unobstructed areas and water surfaces outside hurricane-prone regions. This category includes smooth mud flats, salt flats, and unbroken ice.

#### A.4.2.7 *Seismic design criteria.*

A.4.2.7.1 *Scope.* The seismic design criteria section is adapted from chapter 11 of ASCE 7. The basis for the seismic load requirements can be found in references A.4-6 and A.4-7 (in Cited References section at the end of this appendix).

The conditions permitting an exemption from performing a detailed seismic analysis are consistent with the requirements of ASCE 7, Sec. 11.4.1 and 11.7. The requirements of this standard, which ensure continuous reinforcing in the support wall and a continuous lateral-force-resisting system, satisfy the design and detailing requirements for Seismic Design Category A structures.

A.4.2.7.2 *Site classification.* Table 20.3-1 in chapter 20 of ASCE 7 is the basis for site classification, and is included here as Table A.2 for convenience of the user.

A.4.2.7.3 *Site coefficients.* Determination of the site class for seismic design should be included as part of the geotechnical investigation; see Sec. C.5.4 of appendix C.

A.4.2.7.4 *Ground motions.* Ground motions may be determined using mapped acceleration parameters (Sec. 4.2.7.5.1) or site-specific procedures (Sec. 4.2.7.6). Site-specific procedures are required on Site Class F sites, and the

**Table A.2 Site classification**

Soil Class	$\bar{v}_s$ ft/sec (m/sec)	$\bar{N}$ or $\bar{N}_{ch}$ Blows/ft (Blows/0.3 m)	$\bar{s}_u$ psf (kPa)
A Hard rock	> 5,000 (> 1,520)	Not applicable	Not applicable
B Rock	2,500 to 5,000 (760 to 1,520)	Not applicable	Not applicable
C Very dense soil and soft rock	1,200 to 2,500 (365 to 760)	> 50	> 2,000 (> 100)
D Stiff soil	600 to 1,200 (180 to 365)	15 to 50	1,000 to 2,000 (50 to 100)
E Soft clay soil	< 600 (< 180)	< 15	< 1,000 (< 50)
	Any profile with more than 10 ft (3 m) of soil having the following characteristics:		
	– Plasticity Index $PI > 20$		
	– Moisture content $w \geq 40\%$		
	– Undrained shear strength $\bar{s}_u < 500$ psf (25 kPa)		
F Soils requiring site response analysis in accordance with Sec. 4.2.7.6	Where any of the following conditions is satisfied, the site shall be classified as Site Class F and a site response analysis in accordance with Sec. 4.2.7.6 shall be performed.		
	1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.		
	2. Peats and/or highly organic clay [ $H > 10$ ft (3 m)] of peat and/or highly organic clay where $H$ = thickness of soil.		
	3. Very high plasticity clays [ $H > 25$ ft (7.6 m) with $PI > 75$ ].		
	4. Very thick soft/medium stiff clays [ $H > 120$ ft (37 m)] with $\bar{s}_u < 1,000$ psf (50 kPa).		

site-specific  $MCE_R$  spectral response evaluation should be performed as part of the geotechnical investigation.

A.4.2.7.5 General procedure: risk-targeted maximum considered earthquake ( $MCE_R$ ) and design spectral response.

A.4.2.7.5.1 Mapped acceleration parameters. Ground motion accelerations are based on a 2 percent probability of exceedance in 50 years, equivalent to a recurrence interval of approximately 2,500 years. The accelerations  $S_5$  and  $S_1$  may be determined from ASCE 7, chapter 22, or from the USGS Earthquake Hazards Program Web site. It has a number of methods for determining acceleration parameters based on ASCE 7-10, including a ground motion calculator that is run online: <http://earthquake.usgs.gov/designmaps/us/application.php>.

The program requires latitude/longitude as input to calculate accelerations  $S_S$  and  $S_1$ . The USGS Web site is also a source for the acceleration maps available in PDF format.

The mapped values of  $S_S$  and  $S_1$  are for structures founded on firm rock sites (Site Class B). The short period acceleration  $S_S$  was established as 0.2 seconds, which is representative of the shortest effective period of structures considered in ASCE 7. The two response spectrum parameters,  $S_S$  and  $S_1$ , are sufficient to define an entire response spectrum for the period range of importance for most structures on Class B sites.

**A.4.2.7.5.2 Maximum considered earthquake.** In order to obtain acceleration response parameters that are appropriate for sites with characteristics different than those of Class B sites, it is necessary to modify the  $S_S$  and  $S_1$  values with the site coefficients  $F_a$  and  $F_v$ . These site coefficients scale the  $S_S$  and  $S_1$  values determined for firm rock sites to appropriate values for other site conditions.

**A.4.2.7.5.3 Design spectral response acceleration.** Structural design is performed for earthquakes that are two-thirds of the maximum considered earthquake response spectra. The parameters  $S_{DS}$  and  $S_{D1}$  define the acceleration response spectrum for the design level earthquake as shown in Figure A.1.

**A.4.2.7.6 Site-specific procedure: risk-targeted maximum considered earthquake ( $MCE_R$ ) and design spectral response.** Probabilistic ( $MCE_R$ ) procedures or deterministic ( $MCE_R$ ) procedures may be used in evaluating site-specific maximum considered earthquake. Deterministic procedures are normally used in regions of high seismicity, such as coastal California, where the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of

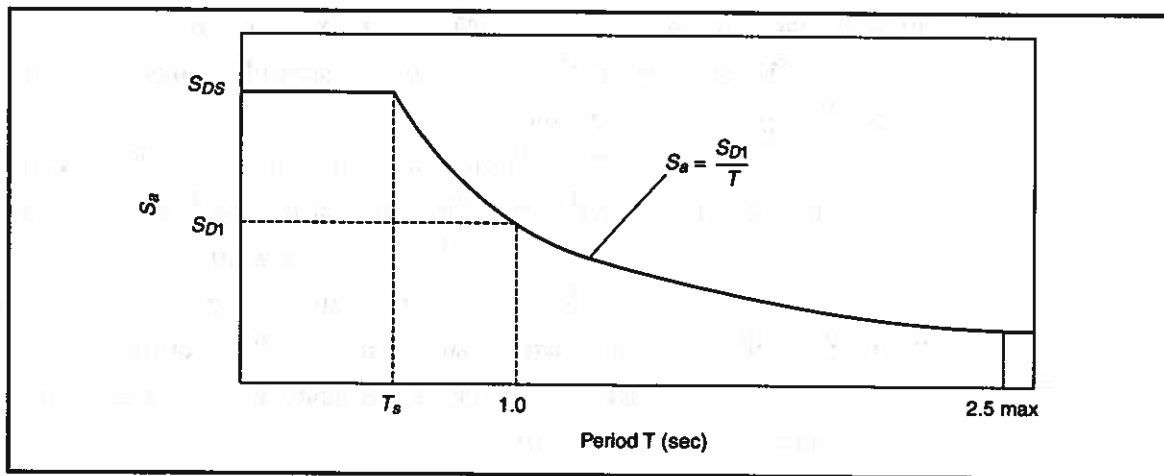


Figure A.1 Design spectral acceleration (Eq 4-11)

well-defined fault systems. The characteristic earthquakes of these defined faults are used to establish the site-specific  $MCE_R$ . In regions where these conditions do not exist, probabilistic procedures are used to determine the site-specific  $MCE_R$ .

A.4.2.7.6.1 Ground motion hazard analysis. Special care must be exercised when generating a design response spectrum from a site-specific spectrum with humps and jagged variations. FEMA 450 requires that the parameter  $S_{DS}$  be taken as the spectral acceleration from the site-specific spectrum at a 0.2-second period, except that it is not to be taken less than 90 percent of the peak spectral acceleration at any period larger than 0.2 seconds. Similarly, the parameter  $S_{D1}$  is taken as the greater of the spectral acceleration at 1-second period or two times the spectral acceleration at 0.2-second period. The parameters  $S_{MS}$  and  $S_{M1}$  are taken as 1.5 times  $S_{DS}$  and  $S_{D1}$ , respectively. The values so obtained are not to be taken as less than 80 percent of the values obtained from the general procedure of Sec. 4.2.7.5. The resulting site-specific design spectrum should be generated in accordance with Sec. 4.2.7.6.3 and should be smoothed to eliminate extreme humps and jagged variations.

A.4.2.7.8 Seismic design category (SDC). The standard incorporates seismic design and detailing requirements of ASCE 7 for major structural components that do not require determination of SDC. Design and detailing of appurtenances such as foundation or anchorage to concrete may require determining SDC for compliance with ASCE 7 or the building code.

#### A.4.2.8 Seismic load.

##### A.4.2.8.2 Seismic load effects.

A.4.2.8.2.1 Vertical. Vertical seismic load effect  $E_v$  occurring in combination with the horizontal seismic load effect  $E_h$  is determined from ASCE 7, Eq 12.4-4. 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 (SRSS) method rather than by direct summation.

A.4.2.8.2.2 Horizontal. Horizontal seismic load effect  $E_h$  occurring in combination with the vertical seismic load effect  $E_v$  may be determined either by the equivalent lateral force procedure or by appropriate alternate procedures.

A.4.2.8.2.3 *P*-delta effects. 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.

### A.4.2.8.3 Seismic base shear.

A.4.2.8.3.1 Equivalent lateral force procedure. Seismic base shear  $V$  using Eq 4-15 is the product of the seismic response coefficient  $C_s$  and the structure gravity load  $W$ . The equivalent lateral force procedure is intended to provide a relatively straightforward design approach where complex analyses, accurately accounting for dynamic and inelastic response effects, are not necessary.

Spectral acceleration  $S_a$  is used in Eq 4-16a in order to accommodate both acceleration parameters determined by site-specific procedures as well as the general procedure. When site-specific procedures are not used, Eq 4-16a can be written as:

$$C_s = \frac{S_{D1}I}{RT} \quad \text{where: } \frac{S_{D1}}{T} \leq S_{DS} \quad (\text{Eq A.4-1})$$

A.4.2.8.3.2 Alternative procedures. 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. The requirements of this section are based on Sec. 15.7.10 of ASCE 7 that permits consideration of soil-structure interaction and fluid-structure interaction in design of elevated tanks.

Soil-structure interaction provisions in ASCE 7 are based on dynamic behavior of soils beneath the foundation at small strain levels, and require knowledge of shear wave velocity of the foundation soils. It is determined by field tests or from empirical relationships. Except when determined by field tests, shear wave velocity used for analysis should not exceed the minimum values associated with the site class in Table A.2.

In order to use fluid-structure interaction in the design, it is necessary to demonstrate the behavior of the sloshing mechanism by analysis or testing. A further requirement is that the period of the sloshing water mass be at least three times greater than the period of the confined liquid considered to be the fundamental period of the structure.

Fluid-structure interaction results from the sloshing that occurs in response to dynamic shaking during an earthquake. This results in a portion of the water mass acting as a rigid mass attached to the structure (impulsive mass), and the sloshing portion (convective mass) responding at a much longer period. The typical composite tank configuration has a cylindrical side shell supported by a cone that is attached to a dome or flat slab floor. For this configuration, it has been demonstrated (reference A.4-4) that the sloshing behavior can be represented with reasonable accuracy by an equivalent cylinder of equal volume. With an equivalent cylinder representing

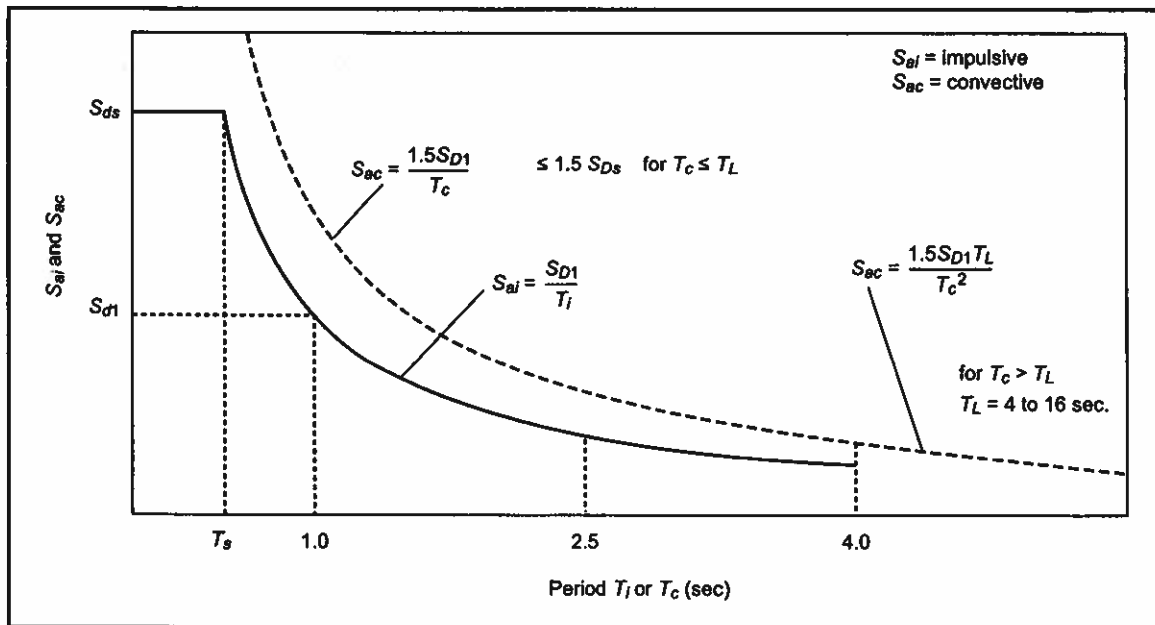


Figure A.2 Impulsive and convective design spectral acceleration

the water mass, the dynamic behavior of the tank can be evaluated using the simple two-mass dynamic analysis presented in references A.4-1 through A.4-3. For the typical composite tank configuration, the equivalent cylinder is considered to have a diameter equal to that of the cylindrical shell, a height calculated to provide a volume equal to the stored water, and top of water surface the same as the actual structure. The design response spectrum, consistent with Sec. 15.7.10.2 of ASCE 7, for impulsive and convective components is shown in Figure A.2. The 1.5 factor included with the convective component that accounts for using 0.5 percent critical damping rather than 5 percent is from reference A.4-5.

The decrease in seismic forces associated with sloshing is most strongly influenced by the diameter-to-height ratio  $(D/H)_e$  of the stored water, which for typical composite tank configurations will range between 1.0 and 3.5. Over this range, the reduction in seismic forces due to the sloshing effect will be 25 percent or less for  $(D/H)_e$  less than 1.0, and up to 50 to 60 percent for the largest  $(D/H)_e$  ratio.

**A.4.2.8.4 Seismic force distribution.** Equation 4-17 is the seismic force distribution prescribed in ASCE 7. 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.

#### A.4.2.8.5 Seismic moment.

A.4.2.8.5.2 Seismic moment at level  $x$ . Equation 4-19b assigns 25 percent of the calculated overturning moment at the base to the top of the concrete support wall. This is a less conservative, but more realistic, variation of the ASCE 7, Sec. 12.2.5.3, requirement for inverted pendulum structures, which requires assignment of 50 percent of the calculated base moment to the top of the structure.

A.4.2.8.6 Response modification coefficient. The response modification factor is an empirical reduction factor intended to reflect ductility, damping, and overstrength of the lateral-load-resisting system.

The lateral-load-resisting system for composite tanks is a circular load-bearing concrete shear wall, and a response modification factor of 3.0 is reasoned to be conservative when compared to equivalent systems for buildings. For bearing wall systems, ASCE 7 provides an R-value of 4.0 for an ordinary reinforced concrete shear wall and an R-value of 5.0 for a special reinforced concrete shear wall. The design and detailing provisions of this standard are comparable to the requirements for special reinforced concrete shear walls.

#### A.4.2.8.7 Structure period.

A.4.2.8.7.1 Fundamental period. 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 Equation C5.2-1 in reference A.4-7) is commonly used:

$$T = 2\pi \sqrt{\frac{\sum (w_i d_i^2)}{g \sum (F_i d_i)}}$$

where:  $d_i$  = 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 Sec. 4.2.8.4. The lumped-mass loads  $w_i$  may be substituted for the applied lateral forces  $F_i$  without significant loss of accuracy.

The fundamental period of vibration  $T$  may be approximated by the following equation when a single lumped mass ( $W/g$ ) is used to represent the mass of the water and the participating dead load.

$$T = 2\pi \sqrt{\frac{(W/g)}{k_c}} \quad (\text{Eq A.4-2})$$

The single lumped mass ( $W/g$ ) comprises the dead load of tank and tank floor, the water load, and not more than two-thirds of the dead load of the support wall. The single-mass approximation assumes a cantilever of uniform stiffness

with the effective structure mass located at the centroid of the stored water. This approximation is reasonable for an elevated water tank where the water weight is typically 80 percent of the total structure weight.

Where the two-mass dynamic analysis for fluid structure interaction described in commentary Sec. A.4.2.8.3.2 is used, the mass for calculating fundamental period includes only the impulsive component of water mass and not the total water mass.

The structure lateral stiffness  $k_c$  is determined from the deflection of the concrete support structure acting as a cantilever beam of length  $l_g$  subjected to a concentrated end load. The flexural stiffness for this condition is

$$k_c = \frac{3E_c I_c}{l_g^3} \quad (\text{Eq A.4-3})$$

The modulus of elasticity of concrete  $E_c$  is determined in accordance with ACI 318, and  $I_c$  is the moment of inertia of gross concrete section of the cylindrical shaft about the 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.

A simple formula to approximate the fundamental period is useful for preliminary design and provides a means of checking that unrealistically large values of fundamental period  $T$  are not used for final design. The requirement adopted here that fundamental period  $T$  not exceed  $C_u T_a$  is modeled after the provisions for buildings in chapter 12 of ASCE 7. The approximate fundamental period  $T_a$  is based on first principles, and the coefficient for upper limit on calculated period  $C_u$  is based on Table 12.8-1 in ASCE 7.

**A.4.2.8.7.2 Approximate fundamental period.** The cross-sectional area of a thin cylinder of thickness  $h_w$  is  $A_c = \pi D_w h_w$ , and its moment of inertia is  $I_c = .125\pi(D_w)^3 h_w = 0.125A_c(D_w)^2$ . Substituting for  $I_c$  in Eq A.4-3 and then into Eq A.4-2 results in the approximate period equation  $T_a$ :

$$T_a = 2\pi \sqrt{\frac{(8W_l l_g^3)}{3A_c E_c D_w^2 g}} \quad (\text{Eq A.4-2})$$

Substituting average concrete compression stress  $f_{ca}$  for the term  $(W_l/A_c)$  and rearranging gives Eq 4-21a. The term  $(f_{ca}/E_c)$  in Eq 4-21a represents the concrete compressive strain that will be in the range of 200 to 300 microstrain. Substituting  $250 \times 10^{-6}$  for  $(f_{ca}/E_c)$ , and  $l_{ga}$  for  $l_g$  results in the simplified Eq 4-21b.

**A.4.2.8.7.3 Coefficient for upper limit on calculated period.** Interpolated values of  $C_u$  in Table 3 can be expressed in equation form by



$$C_u = 1.9 - 2S_{D1} \leq 1.7 \quad \text{for } S_{D1} \leq 0.2$$

$$C_u = 1.7 - S_{D1} \geq 1.4 \quad \text{for } S_{D1} > 0.2$$

A.4.2.8.8 *Sloshing.* Consideration of the sloshing wave due to seismic acceleration is in accordance with requirements of ASCE 7. Equation 4-23a may be used in calculating sloshing wave height for any convective period  $T_c$  since it provides conservative results when the convective period  $T_c$  is longer than the long-period transition period  $T_L$ . Figure 22-12 from ASCE 7 shows  $T_L$  to be 4, 6, 8, 12, and 16 seconds in various parts of the conterminous United States, and since the convective period  $T_c$  will almost always be less than 7 seconds for composite tanks, Eq 4-23b will only be applicable to areas where  $T_L$  is 6 seconds or less.

#### A.4.2.9 *Other loads.*

A.4.2.9.1 *Vertical load eccentricity.* Eccentricity of dead and water loads causes additional overturning moments that should be accounted for in the design. Eccentricity occurs when: (1) the tank is not concentric with the support wall, (2) the support wall is out-of-plumb, or (3) the foundation tilts because of differential settlement. The total eccentricity included in Eq 4-25a 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 4-25a should not be used as construction tolerances.

It is assumed that half the minimum eccentricity in Eq 4-25a is due to tilting of the foundation (foundation tilt of  $\frac{1}{800}$  equal to 0.0716 degrees). 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 4-25b.

A.4.2.9.2 *Construction loads.* The structure must be checked for the effects of any construction-related loads that are applied during the construction process. Accommodating these loads might require the addition of local reinforcement that exceeds that required for service loads. Additionally, construction loads can occur before concrete has attained its specified  $f'_c$  or before all the components of the concrete support structure have been constructed.

### Sec. A.4.3 **Analysis and Design**

A.4.3.1 *Analysis.* The purpose of the selected method of analysis is to determine the applicable axial loads, shears, and moments that are to be used in the

design of the composite elevated tank and its components, and not to substantiate the use of loads that are less than those specified by the standard.

**A.4.3.3 Unfactored load combinations.** Unfactored service load combinations are presented in a form comparable to factored loads. 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.

**A.4.3.4 Factored load 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 in this standard is 1.5. This is intermediate between basic load factors of 1.2 for dead load and 1.6 for live load. When using appendix C of ACI 318-02 or earlier editions of ACI 318 that require basic load factors of 1.4 and 1.7 for dead and live loads, respectively, the comparable load factor for water is 1.6 based on a comparison of load factor /  $\phi$ .

The load combinations in this standard conform to ACI 318, except for combinations U1.4 and U2.2, which include seismic loads and are based on ASCE 7 load combinations. Load combination U1.4 is a variation of the ASCE 7 basic load combination  $1.2D' + 1.0E' + 0.5L + 0.2S$ . The term  $D'$  includes  $D + F$ ; eccentricity  $G$  is substituted for  $L$ , which is instead assigned a more conservative load factor of 1.0; 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  $E_h$  and vertical seismic load  $E_v$ . Vertical seismic load effects have generally not been included in design of elevated tanks in the past, and the ability to continue to exclude them is accomplished by treating vertical seismic load effects as a separate load component  $E_v$ . The structural effects  $T$  of differential settlement, creep, shrinkage, or temperature effects are usually not significant and have not been shown for clarity.

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## SECTION A.5: STEEL TANK

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### Sec. A.5.1 General

A.5.1.3 *Configuration.* Typically a steel tank is composed of a connecting element to the concrete support structure, a conical shell outside the concrete support, a cylindrical shell that supports the outer edge of the roof, and a liner or structural bottom.

### Sec. A.5.2 Material Requirements

A.5.2.2 *Steel plate.* Table 4 lists material that may be used for elevated tanks. These materials are permitted without regard to design metal temperature because tension stress limits are relatively low. Other plate material is permitted when specified provided that consideration is given to design metal temperature and weldability.

Thickness limit of 4 in. (100 mm) in compression represents the practical maximum size anticipated. Smaller values for ASTM A537 and A573 steels are the maximum thickness produced under the specification for the class or grade. Thickness limits are not applicable to flexural elements such as base plates when in-plane membrane stresses are not significant; less than 6,000 psi (41 MPa).

Table A.3 provides additional reference information for steel plate listed in Table 4 that may be required for weld design or evaluating tolerances.

A.5.2.3 *Other structural steel.* Other steel tank materials may include rolled structural shapes, hollow structural sections, bar and plate for fabricated sections, bolts, anchor bolts, and threaded rods.

### Sec. A.5.3 Analysis and Design

A.5.3.1 *Analysis.* Membrane theory of thin elastic shells is applicable where bending stiffness is sufficiently small that the resulting flexural strain is not significant, or where the shells are flexurally stiff but loaded and supported in a manner that avoids the introduction of significant flexural strain. Membrane analysis includes analysis of boundary elements such as tension or compression rings that resist unbalanced edge forces at shell discontinuity junctures.

Design using membrane theory only is applicable to tank configurations with a history of satisfactory performance. For other or new configurations, procedures based on strain compatibility should be used for analysis of shell discontinuity junctures where bending stresses may be significant. Classical shell theory, simplified

Table A.3 Plate material properties

Specification	Grade	Minimum Strength (US customary)			Minimum Strength (SI units)			ASME* P & Group Numbers			AWS B2.1† Group Number
		Yield (psi)	Tensile (psi)	Yield (MPa)	Tensile (MPa)	General Requirements	P-number	Group Number			
ASTM A36	—	36,000	58,000	248	400	ASTM A6	1	1	1	1	
ASTM A516	60	32,000	60,000	221	414		1	1	1	1	
	65	35,000	65,000	241	448	ASTM A20	1	1	1	1	
	70	38,000	70,000	262	483		1	2	2	1	
ASTM A537	CL 1	50,000	70,000	345	483	ASTM A20	1	2	2	2	
ASTM A572	42	42,000	60,000	290	414	ASTM A6	NL	NL	NL	1	
	50	50,000	65,000	345	448		NL	NL	NL	1	
ASTM A573	58	32,000	58,000	221	400		NL	NL	NL	1	
	65	35,000	65,000	241	448	ASTM A6	NL	NL	NL	1	
	70	42,000	70,000	290	483		NL	NL	NL	2	
CSA G40.21	260W	38,000	60,000	262	414		1	1	1	1	
	300W	44,000	65,000	303	448	CSA G40.20	NL	NL	NL	1	
	350W	50,000	65,000	345	448		NL	NL	NL	2	

\* From ASME Boiler and Pressure Vessel Code, Section IX, QW-420. "NL" indicates the material is not listed. The P-number is for weld procedure grouping, and the group number is for specified impact test requirements.

† From AWS B2.1, Annex D, Base Metal Grouping. All listed plate is Material Number 1.

mathematical models, or numerical solutions using finite element, finite differences, or numerical integration techniques can be used to determine in-plane membrane forces, shear, and flexure. An equivalent stress formulation, such as the von Mises stress intensity, should be used to evaluate calculated working stresses based on these procedures. Maximum equivalent surface stress should be limited to 60 percent of the yield strength for gravity loads, and 80 percent of the yield strength for load combinations including wind or seismic forces. The membrane component of calculated stress should not exceed allowable stresses of Table 5. Higher stresses, including localized yielding, are acceptable near discontinuity junctures when it can be demonstrated that higher stresses do not lead to progressive collapse of the overall structure, and that general stability is maintained. Weldments located within a distance  $0.78\sqrt{R_t}$  of the discontinuity and subject to significant stress should be designed for cyclic loading. Membrane component of stress should be in accordance with allowable stresses in Table 5.

#### A.5.3.2 *General design requirements.*

A.5.3.2.3 Corrosion allowance. The need for corrosion allowance is dependent on the potential for lack of maintenance of coatings or other protection systems. When required, corrosion allowance for the steel tank or specific components is typically specified as 0.0625 in. (1.59 mm).

#### A.5.3.3 *Design of welds.*

A.5.3.3.2 Allowable stress. Weld allowable stresses are based on AWS D1.1 requirements for statically loaded structures.

A.5.3.3.3 Joint efficiency factors. Joint efficiency factors less than 1.0 used in the standard reflect the level of inspection specified in Sec. 9.5.2.

A.5.3.3.4 Effective area of welds. The standard defines joint requirements for partial penetration welds with bevel or V-joints having an included angle of 45 degrees to 60 degrees. Refer to Sec. 2.3 of AWS D1.1 for other joint configurations.

#### A.5.3.3.5 *Weld joint details.*

1. Fillet welds or partial penetration welds are not intended for use in connecting primary stress joints in shell plate; see Sec. 5.3.3.5 (item 1). Allowable stress for fillet welds in Sec. 5.3.3.2 applies only to structural welds that are not part of the water-containing structure, such as longitudinal welds for roof rafters and attachments.

2. The default requirement is to seal-weld all exposed joints. This approach prevents corrosion of steel surfaces that are inaccessible for painting or maintenance and may otherwise corrode. Examples include roof interior lap joints, rafter connections, attachments, and penetrations.

3. Minimum weld sizes in Table 6 are based on requirements of AWS D1.1, except minimum weld size of  $\frac{3}{16}$  in. (4.76 mm). AWS D1.1 permits  $\frac{1}{8}$  in. (3.18 mm) minimum weld size for base metal thickness to  $\frac{1}{4}$  in. (6.35 mm) inclusive. The use of  $\frac{3}{16}$  in. (4.76 mm) minimum weld size is consistent with tank construction industry practice for nonseal welds.

4. Welding to steel elements embedded in concrete requires details and procedures that minimize the effects of heat, restraint, and shrinkage. Heat from welding to steel embedments and weld shrinkage can cause spalling or cracking of concrete. Where long or heavy welds are required, the weldment should be made on a projecting element of the embedment away from the concrete surface.

#### A.5.3.4 *Design of shell plates and structural components.*

A.5.3.4.1 General. Shell plate and tension/compression ring allowable stresses in Table 5 are applicable to membrane theory of analysis discussed in Sec. A.5.3.1.

A.5.3.4.2 Shell plates. When the average thickness of cylindrical shell plates is greater than that required by the following equation, intermediate stiffeners are not required to resist external wind pressure. The equation is based on a simplification of the David-Taylor Model Basin formula using a factor of safety of 1.0 (reference A.5-2). The critical wind-buckling condition is represented by the stagnation pressure acting as a uniform external pressure at an angle of zero degrees from the wind for a shell that buckles into 18 nodes or circumferential waves. R.V. McGrath's original paper (reference A.5-2) recommended using external pressure  $p_z = 0.002558V^2K_zG + p_v = 36$  psf (1,720 Pa), where wind velocity  $V$  is 100 mph (45 m/sec), height factor  $K_z$  is 1.1, gust factor  $G$  is 1.1, and vent vacuum setting  $p_v$  is 5 psf (240 Pa). The use of pressure  $p_z = 36$  psf (1,720 Pa) has given satisfactory performance in nonhurricane regions.

$$t_a = 0.9D \left[ \frac{p_z}{E_s} \frac{h}{D} \right]^{0.4} \quad (\text{consistent units})$$

where:

$t_a$  = average shell thickness between stiffeners

$D$  = shell diameter

$h$  = shell height between stiffeners

$E_s$  = modulus of elasticity of steel plate

$p_z = q_s I G_f K_z + p_v$  = local wind plus vacuum pressure acting on the shell;  
 $q_s I G_f K_z$  is determined from Sec. 4.2.6.3, and  $p_v$  is the vent vacuum setting; 5 psf (240 Pa) minimum is recommended.

A.5.3.4.4 Access tube. The David-Taylor Model Basin formula (in reference A.5-3) is an acceptable method for calculating the critical buckling pressure caused by external water pressure acting on shell plate. The standard does not provide specific requirements for analysis and design of access tube shell plate or stiffening. References A.5-3, A.5-4, and A.5-5 may be used for guidance in design of these elements.

A.5.3.4.5 Tank roof. Roof supports are designed using the allowable stresses of ANSI/AISC 360 and the unfactored load combinations in Sec. 4.3.3. Alternatively, roof supports may be designed using the strength limit states of AISC 360 and the factored load combinations in Sec. 4.3.4. Minimum slope of roofs should not be less than 3/4:12.

Unstiffened plate roofs may be used for small-diameter tanks. Design requirements can be found in API 650.

Rafter and stiffener details should be configured to provide proper access for application and maintenance of coatings. Locations that are difficult to paint and maintain can cause premature coating failures and corrosion.

A.5.3.4.6 Steel tank connection. Connection strength of the steel tank and concrete support structure is checked for seismic loads assuming elastic behavior using  $R/I$  of 1.0. This is to preclude a possible connection failure during a seismic event.

A.5.3.4.7 Liner plate. Welded liner plates over concrete floors are non-structural elements where coatings, welds, and connecting elements may be damaged by deformations resulting from improper support. To function properly, the liner plate must conform to the shape of the underlying concrete floor. This is achieved through a combination of controlling geometric tolerances in concrete construction and steel fabrication, and workmanship in fitting and welding of the liner plate. Where liner plate does not conform to the supporting concrete floor, the space between liner plate and concrete floor should be filled with grout. Where grouting is intended for liner plate support, provide 1-in. (25-mm) minimum grout space.

#### A.5.3.5 Meridional compression strength of shells.

A.5.3.5.1 General. The equations used to define the limiting meridional compression strength  $F_L$  are based on reference A.5-1. The provisions are applicable to shells subjected to meridional compression stress combined with either no stress or tension in the circumferential direction. ASME Boiler and Pressure Vessel Code,

Section VIII, may be used to design shells subjected to circumferential compression combined with meridional tension or compression.

Table 7 lists radius-to-thickness ratios  $(R/t)_c$  for various yield strengths that are calculated by equating Eq 5-3a and Eq 5-3b. This represents the transition point between inelastic and elastic portions of the meridional buckling strength equations.

The standard provides two methods for calculating the meridional compression strength  $F_L$  based on level of design analysis and field inspection.

A.5.3.5.2  $F_L$ : method 1. Meridional compression strength  $F_L$  determined by this method is applicable to all shells. It is a tracking equation that reflects historical practice using only membrane analysis for design and requires only visual inspection for tolerances. Analysis using the procedures of method 2 is recommended for new configurations without a proven history.

A.5.3.5.3  $F_L$ : method 2. This method usually results in higher allowable design stresses than permitted by method 1. Extensive analytical verification and quality assurance and control are required to ensure a comparable level of safety. Meridional compression strength  $F_L$  used for design is the smaller of critical buckling strength  $F_{cr}$  from a nonlinear buckling analysis and maximum meridional compression strength  $F_{xe}$  (Eq 5-6). Equation 5-6 is a floor and a check of the analytical results. It is from reference A.5-1 and is based on testing and extensive computer buckling analyses.

Shell buckling is sensitive to imperfections, support conditions, and loads that cause stress and deformation. The nonlinear buckling analysis requires the designer to have a complete understanding of the behavior of the structure. Proper modeling is critical in achieving this goal and should only be performed by qualified analysts. Field measurements defined in Sec. 9.4.2 are required to verify that construction tolerances are consistent with assumptions used in the analysis.

Determination of plasticity reduction factor  $\eta$  for use in Eq 5-6 requires a trial procedure using Equations 5-8 through 5-10. One approach is the following:

1. Assume plasticity reduction factor  $\eta$  is 1.0 to calculate a trial value for  $F_{xe}$ .
2. With trial value  $F_{xe}$ , calculate  $F_{eff}$  from Eq 5-10,  $\Delta$  from Eq 5-9,  $\eta$  from Eq 5-8, and new value of  $F_{xe}$  from Eq 5-6.
3. Compare the trial value of  $F_{xe}$  to the new calculated value. If the difference is within an acceptable margin, the solution is achieved. If not, repeat the procedure until a solution is achieved. Using the average of the trial and new



calculated values for the next iteration, in lieu of guessing, should minimize the number of trial solutions.

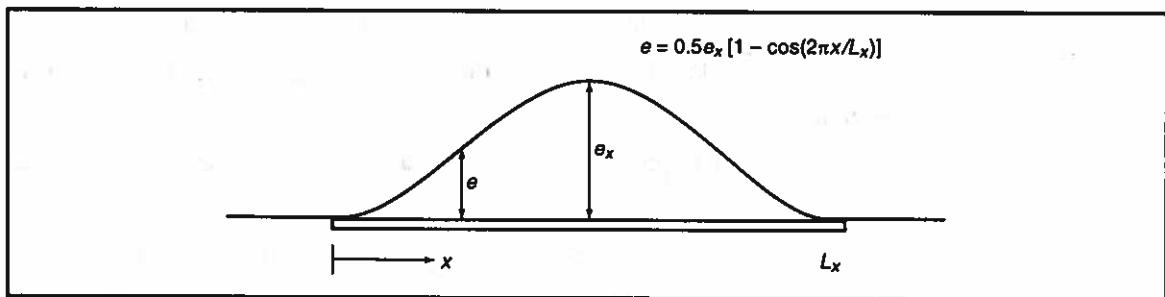
Table 8 is presented in the standard with numerical values only for the materials listed in Table 4. The equation forms of the Table 8 coefficients are given below for use with materials having other yield strengths or for use in computer programs.

$$\begin{aligned}
 A &= 0.651 + 0.0000038F_y \quad (\text{psi units}) & A &= 0.651 + 0.00055F_y \quad (\text{MPa units}) \\
 B &= 0.355 - 0.00000292F_y \quad (\text{psi units}) & B &= 0.355 - 0.00042F_y \quad (\text{MPa units}) \\
 D_1 &= \frac{F_y}{136,000} + 0.196 \quad (\text{psi units}) & D_1 &= \frac{F_y}{937.7} + 0.196 \quad (\text{MPa units}) \\
 D_2 &= \frac{118.34}{F_y^{0.0573}} - 60.17 \quad (\text{psi units}) & D_2 &= \frac{88.98}{F_y^{0.0573}} - 60.17 \quad (\text{MPa units})
 \end{aligned}$$

A.5.3.5.4 Critical buckling strength,  $F_{cr}$ . The modeled imperfection used in the nonlinear buckling analysis should reflect the as-built configuration of the shell. The modeled imperfection cannot be smaller than permissible tolerances specified in Sec. 5.4.4.1, but its shape is not explicitly defined. An acceptable shape, based on the modeled imperfection used in reference A.5-1, is shown in Figure A.3.

Critical failure load may be determined by incrementing meridional force, incrementing fluid density, or both. A conservative approach is to use the smaller load determined by both methods. In regions of low seismic risk, the designer can make the determination as to which loading type is appropriate for the structure. This option is not available in higher seismic regions because seismic acceleration has the effect of increasing fluid density.

Boundary conditions can have a significant effect on failure load. The pinned base connection should always be checked as a lower bound case of critical buckling load.



Source: From reference A.5-1.

Figure A.3 Modeled imperfection

### Sec. A.5.4 Fabrication and Construction Requirements

This section incorporates various provisions of AWS D1.1 by reference, which contains requirements for statically loaded structures and cyclically loaded structures. Only the provisions for statically loaded structures are applicable to this standard.

A.5.4.1.1 Weld procedure specifications. Weld procedure specifications (WPSs) are the basis for all welding on the steel tank. Procedure qualification records (PQRs) define welding variables applicable to the welding process. A WPS may require the support of more than one PQR, while one PQR may support a number of WPSs. AWS standard weld procedure specifications complying with AWS B2.1 are permitted, and these are supported by PQRs maintained at the Welding Research Council. The basis for establishment of AWS standard WPSs is described in Annex A of AWS B2.1.

#### A.5.4.3 *Welding operation.*

A.5.4.3.1 Weld quality. Applicable weld procedure specifications should be maintained at the site during welding operations for reference. Welding personnel should be informed of their proper use and the applicable WPSs to be followed.

A.5.4.3.3 Welding environment. Welding is not permitted when the surfaces of the parts to be welded are wet from rain, snow, or ice; when rain or snow is falling on such surfaces; or during periods of high winds, unless the welder or welding operator and the work are properly protected. The requirement for maintaining an environment at the weldment above 0°F (-20°C) is for providing conditions under which the welding operator can produce optimum results, and shelters or enclosures are necessary in colder conditions. The welding environment provisions do not alter preheat or interpass temperature requirements of Sec. 5.4.3.6 (item 1).

A.5.4.3.4 Temporary and tack welds. Clips, jigs, lugs, and other devices welded to plates for erection purposes must be removed without damage to the plates, repaired if necessary, and the surface ground smooth and flush.

A.5.4.3.5 Control of distortion and shrinkage. Procedures that minimize distortion from weld shrinkage include the following:

1. A weld sequence that balances applied heat of welding while welding progresses.
2. Progression of welding from points where parts are relatively fixed in position toward points having greater relative freedom of movement.

**Table A.4 Preheat requirements for standard weld procedure specifications\***

Welding Process for Steel Plate Listed in Table 4	Thickness of Thickest Part at Point of Welding		Minimum Preheat and Interpass Temperature <sup>†</sup>	
	(in.)	(mm)	(°F)	(°C)
Category A: SMAW with other than low-hydrogen electrode	To ¾ inclusive	To 19 inclusive	32	0
	¾ thru 1½ inclusive	19 thru 38 inclusive	150	65
	1½ thru 2½ inclusive	38 thru 64 inclusive	225	107
	Over 2½	Over 64	300	149
Category B: SMAW with low-hydrogen electrode SAW, GMAW, FCAW	To ¾ inclusive	To 19 inclusive	32	0
	¾ thru 1½ inclusive	19 thru 38 inclusive	50	10
	1½ thru 2½ inclusive	38 thru 64 inclusive	150	65
	Over 2½	Over 64	225	107

\* Table adapted from AWS D1.1, Table 3.2.

† When the base metal temperature is below 32°F (0°C), the base metal shall be preheated to a minimum of 70°F (21°C) and the minimum interpass temperature shall be maintained during welding.

**A.5.4.3.6 Preheat and interpass temperature.** Preheat and interpass temperature requirements are an integral part of the procedure qualification records (PQRs) that are the basis for required weld procedure specifications. A change in interpass temperature in excess of 100°F (38°C) requires a new PQR to substantiate the change.

Preheat and interpass temperature requirements for standard weld procedure specifications are listed in AWS D1.1, Table 3.2, and are shown in Table A.4 for materials listed in Table 4 of this standard. These tabulated values are recommended as a guideline for other weld procedure specifications. Category B electrode is recommended for welding plate with thickness greater than 1.5 in. (38 mm).

#### **A.5.4.4 Tolerances.**

**A.5.4.4.1 Local imperfections.** Nonconforming shells where local buckling controls may be checked by performing a nonlinear buckling analysis that is based on the actual imperfections. The analysis should demonstrate a minimum factor of safety of 2 for local buckling.

**A.5.4.5 Painting.** Typically the tank will be hydrotested after painting. This sequence eliminates the additional cost and environmental concerns associated with the disposal of hydrotest water.

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## SECTION A.6: CONCRETE SUPPORT STRUCTURE

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### Sec. A.6.3 Analysis and Design

A.6.3.1 *Analysis.* The provisions of chapter 19 of ACI 318 are the basis for analysis of the support wall and dome tank floor of the concrete support structure. Methods of analysis may be based on 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.

A.6.3.2.2 *Minimum flexural reinforcement.* The minimum flexural reinforcement ratio for the tension face is the same as required by ACI 318 and is intended to prevent abrupt strength changes at the onset of cracking.

A.6.3.2.3 *Minimum axial tension reinforcement.* The minimum reinforcement ratio 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  $7.5\sqrt{f'_c}$ . This requirement is intended to prevent abrupt strength changes when cracking occurs.

A.6.3.2.4 *Spacing of tension reinforcement.* Reinforcement spacing limits apply to regions of significant flexure or direct tension in the wall, tank floor, and ringbeam. The provisions for maximum spacing of flexural reinforcement are based on the requirements of ACI 318. The smaller spacing for axial tension reinforcement is based on the ratio between crack widths for axial tension and flexural tension described in ACI 224.2R.

#### A.6.3.3 *Support wall design.*

A.6.3.3.1 *Design requirements.* Minimum specified concrete strength for durability in Sec. 6.4.2 is 4,000 psi (28 MPa). The maximum specified strength of 6,000 psi (41 MPa) for design is based on experience.

A.6.3.3.2 *Minimum wall thickness.* The 8-in. (200-mm) minimum wall thickness is based on consideration of placing and consolidating concrete in forms.

A.6.3.3.3 *Reinforcement.* Minimum requirements for wall reinforcement in Table 11 are the same as ACI 318 for horizontal reinforcement and 25 percent greater for vertical reinforcement. The requirement in Sec. 6.3.3.8.2 for greater minimum reinforcement when seismic shear  $V_{uw}$  exceeds  $\sqrt{f'_c}A_{cv}$  is based on the shear wall requirement of chapter 21 of ACI 318.

A.6.3.3.4 Axial load and flexure. The general design procedure is to determine maximum loads for a unit width of wall and to design this section as a beam-column in accordance with applicable provisions of ACI 318. Flexure may be ignored when the resultant of the vertical load is within the middle third of the wall ( $e < h_w/6$ ), similar to the empirical design method for walls in ACI 318.

Global slenderness effects (i.e., buckling of the cylindrical wall acting as a cantilever) will not be significant except for very tall small-diameter walls. The standard includes a reduction for local buckling of the wall by the slenderness reduction factor  $\beta_w$ . It is obtained by equating the classical elastic buckling strength of a cylinder,  $\sigma_{cr} = \gamma C_c E_c (2h/D_w)$ , to  $\beta_w f'_c$ , where  $C_c$  is 0.6 for a cylinder and  $\gamma$  is a reduction factor of approximately  $1/7$ .

A.6.3.3.5 Axial compression strength. Nominal axial compression strength is based on ACI 318 requirements for walls and tied columns. Typically the strength of the wall is assessed by Eq 6-4a at locations where flexural loads are small. Significant through-thickness flexure is usually found only at section changes, geometric imperfections, and other discontinuities.

The strength of tied longitudinal reinforcement in Eq 6-4b can be included without regard to minimum reinforcement ratio. However, it is recommended that the contribution of tied longitudinal reinforcement be neglected as compression reinforcement unless the reinforcement ratio is 0.40 percent or greater.

A.6.3.3.6 Circumferential bending moment. The circumferential wind moment coefficient in Eq 6-5 is based on analysis of the wind pressure distribution in ACI 334.2R.\* It is not necessary to combine circumferential wind effects with other loads when determining the requirements for horizontal reinforcement.

A.6.3.3.7 Radial shear. Transverse shear forces occur at radial concentrated loads and adjacent to restrained boundaries such as the top of foundation and the tank floor.

A.6.3.3.8.1 Tangential shear force. Global wind or seismic forces cause tangential shear. Finite element or similar analysis is required where openings remove a significant portion of the wall. The following simplified procedure is permitted when the wall opening ratio  $\Psi$  does not exceed 0.5. Minimum wall diameter to meet this criterion for typical vehicle and personnel door installations is tabulated below.

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\* ACI 334.2R, Reinforced Concrete Cooling Tower Shells—Practice and Commentary, American Concrete Institute, Farmington Hills, MI.

Truck Door Width	Cumulative Opening Width $B_x$ (Including 3.28 ft [1.0 m] Personnel Door)	Minimum Wall Diameter
10.0 ft (3.0 m)	13.28 ft (4.0 m)	34.3 ft (10.5 m)
12.0 ft (3.7 m)	15.28 ft (4.7 m)	39.4 ft (12.0 m)
14.0 ft (4.3 m)	17.28 ft (5.3 m)	44.6 ft (13.6 m)

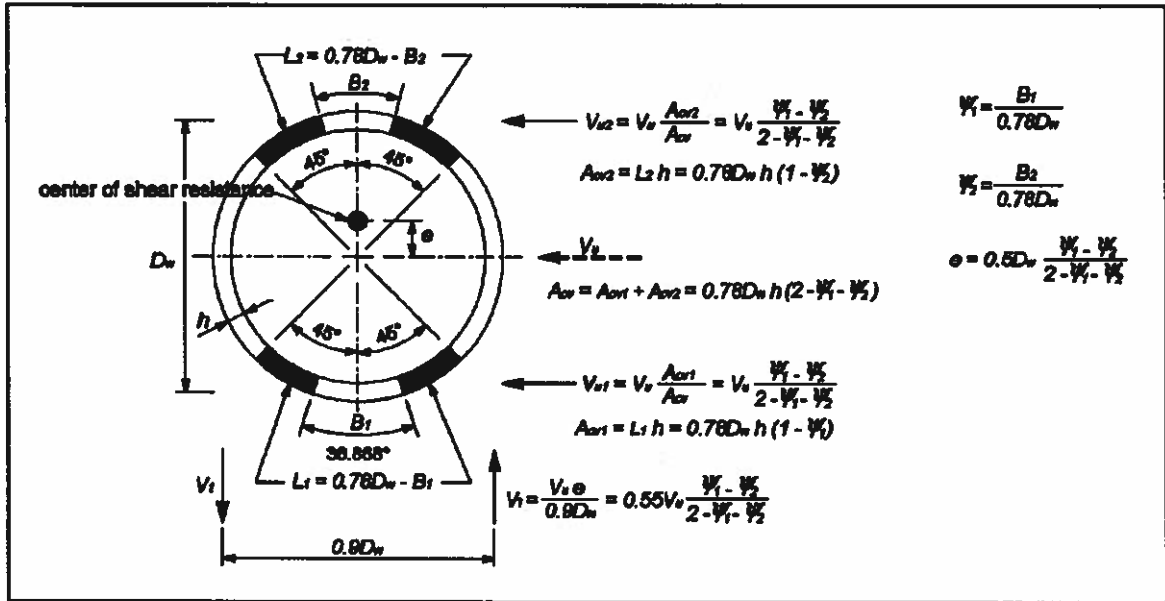


Figure A.4 Total shear,  $V_u$ , distributed to each shear wall

The following simplified procedure may be used to design for tangential shear when the wall opening ratio  $\Psi$  does not exceed 0.5. Tangential shear is assumed resisted by two parallel shear walls of length  $0.78D_w$ . The design shear force per unit of shear wall length is equal to the maximum in-plane shear in a cylinder without openings  $2V_u/\pi D_w$ . The total shear  $V_u$  is distributed to each shear wall in proportion to the relative stiffness of each shear wall as illustrated in Figure A.4. Any torsional eccentricity that may result is assumed to be resisted by the couple that can develop in the transverse sections of wall not directly resisting tangential shear.

**A.6.3.3.8.2 Tangential shear strength.** Generally walls should be proportioned such that factored shear force is not greater than  $2\sqrt{f'_c}A_{cv}$ . This eliminates the need for shear reinforcement greater than minimum requirements.

The equation for nominal shear strength  $V_n$  is 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 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 shear wall width above the base. The second criterion is consistent with ACI 318, and the first ensures that shear across openings is checked.

**A.6.3.4 *Dome tank floor design.*** Concrete domes are designed as shells having in-plane compression forces, except near the inner and outer edges where edge forces and deformation incompatibilities with attached elements occur. The minimum flexural reinforcement requirement (Sec. 6.3.2.2) is intended to ensure adequate stiffness at cracked sections. At other sections, a minimum steel ratio of 0.0018 in each face is required.

**A.6.3.5 *Slab tank floor design.*** Slab floors should be designed to minimize the effects of edge rotation on the tank cone and wall. A conservative approach for calculating deformations is to use the gross concrete area and one-half the modulus of elasticity of concrete.

**A.6.3.6 *Interface region design.*** The conditions at the interface are sensitive to tank and support wall proportions. A series of analyses of the interface region should be performed to investigate the effects of material properties, volume change (creep and shrinkage), construction tolerances, variable water load, and environmental loads.

Axial tension stress in circumferential reinforcement should be limited to 15,000 psi (103 MPa) to prevent excessive hoop deformation and cracking in the interface region.

**A.6.3.7 *Design of wall openings.***

**A.6.3.7.3 Effective beam and column method.** Equation 6-11 is based on deep-beam theory, and was developed around the use of reinforcement with 60,000-psi (414-MPa) yield strength. The 0.14 moment coefficient includes an increase in the average load factor from 1.5 to 1.7.

**A.6.3.7.4 Pilasters.** Pilasters are recommended at large openings such as truck doors where congestion of reinforcement occurs. Pilasters should extend beyond the opening for load transfer into the wall. Out-of-plane forces and deformations should be considered in the design. Forces and deformations in this area are best evaluated with finite element analysis.

## Sec. A.6.4 Fabrication and Construction Requirements

A.6.4.1 *General.* The standard uses the construction provisions of ACI 318 by reference with modifications to define requirements for concrete construction. These requirements are not intended to be a construction specification.

A.6.4.3 *Details of reinforcement.* Concrete cover is measured in rustications or between raised portions of the support wall where it is thinnest.

A.6.4.4 *Curing and protection.*

A.6.4.4.1 *General.* Curing provisions are based on recommendations of ACI 308, which recommends curing for either 7 days at 50°F (10°C) or attaining 70 percent of the specified compressive strength  $f'_c$ . The strength criterion is used in the standard because of its simplicity, and it reflects common practice.

A.6.4.4.2 *Cold weather protection.* The compressive strength requirement for discontinuing protection against freezing is based on attaining sufficient strength to prevent damage from exposure to subsequent freezing temperatures for concrete exposed to weather. Insulating wall forms is common practice when freezing temperatures are expected, and is generally sufficient protection for moderately cold conditions.

A.6.4.4.3 *Hot weather protection.* ACI 305 provides detailed recommendations for hot weather concreting practice. Difficulties may be encountered when concrete placement temperatures approach 90°F (32°C). Set-retarding admixtures or other controls may be necessary when placing concrete at elevated temperatures.

A.6.4.5 *Formwork.* ACI 347R provides recommendations for formwork design, construction, and materials. Support structure forms are typically reused a large number of times, and a durable facing material is required in order to maintain uniformity of the surface.

A concrete compressive strength of 800 psi (5.5 MPa) is generally adequate to prevent damage to concrete surfaces during form removal. Higher concrete compressive strength may be required for subsequent loads, supports, embedments, or construction activities. Wall forms are typically removed the day following a concrete placement after 12 hours of curing. Dome forms generally may be removed after 24 hours and flat slab forms at 72 hours when temperatures are above 50°F (10°C). Verification of concrete strength is recommended for form removal when a minimum compressive strength is required.

A.6.4.7 *Finishes.* The exterior exposed wall surfaces commonly have a light sand-blasted finish that provides a more uniform finish than the as-cast form finish only.



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## SECTION A.7: FOUNDATIONS

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### Sec. A.7.1 General

A.7.1.3 *Geotechnical investigation.* A geotechnical investigation and recommendations are normally based on a limited amount of sampling. The geotechnical engineer should verify during construction that actual conditions are consistent with those that form the basis of the recommendations.

### Sec. A.7.2 Material Requirements

A.7.2.2 *Piles.* Pile materials referenced in ASCE 20 include timber, concrete, and steel.

### Sec. A.7.3 Analysis and Design

#### A.7.3.1 *Sizing foundations.*

A.7.3.1.1 *Strength.* Factors of safety in Table 12 reflect the degree of uncertainty associated with the methods used to determine ultimate capacity. Ultimate capacity is determined by generally accepted geotechnical and civil engineering principles in conjunction with a geotechnical investigation, load tests, or other data.

A.7.3.1.2 *Settlement.* The standard provides performance requirements for sizing foundations for settlement. The combined concrete foundation and support wall are relatively rigid and can undergo significant total settlement without distress. Appendix C provides guidance with respect to settlement limits.

A.7.3.1.3 *Overturning stability.* Resisting moment is the product of gravity load times the distance from the foundation centerline to centroid of soil or pile resistance. The centroid of soil or pile resistance is determined from the contact area or pile group required to resist the gravity load at ultimate capacity. The tension capacity of piles may be considered in assessing the stability ratio. Where high groundwater occurs, the effects of buoyancy shall be included.

A.7.3.1.4 *Lateral resistance.* Shallow foundations are generally capable of resisting lateral loads by base friction or adhesion alone. Lateral loads for deep foundations are generally assumed to be resisted entirely by the piles through shear and flexure and lateral soil resistance. Analysis utilizing "*p-y* curves" provides a reasonable assessment of pile behavior when soil properties are known.

#### A.7.3.2 *Structural design of concrete foundations.*

A.7.3.2.1 *General.* Shallow annular ring foundations are normally proportioned such that the centroid of the annular ring coincides with the support structure wall radius. Otherwise, torsion and biaxial bending effects must be considered.

Significant in-plane shear and moment may result from lateral loads in annular ring pile caps. Narrow, large-diameter pile caps may require a diaphragm to distribute the load to piles. The effects of pile-head fixity should be considered in the design of pile caps.

**A.7.3.3 Structural design of piles.** Table A.5 lists allowable unit stresses for piles based on IBC 2012. ASCE 20 provides guidance for structural design of piles. ACI 336.3R provides recommendations for design and construction of drilled piers. Seismic detailing requirements for piles are found in Sec. 12.13.6 and 14.2.3 of ASCE 7.

**A.7.3.4.1 Minimum foundation depth.** Minimum foundation depth should be increased where frost penetration deeper than shown in Figure 1 may occur. Other considerations such as piping, uplift, or soil bearing capacity may require greater foundation depths.

#### Sec. A.7.4 Fabrication and Construction Requirements

**A7.4.1.1 Excavation.** A lean concrete layer (mud mat) under foundations is recommended for all excavations. It provides a good working surface for placing reinforcement and protects the bearing surface.

**Table A.5 Allowable stress in piles typically used\***

Pile Type and Loading Condition	Maximum Allowable Stress
a. Concrete or grout in compression <sup>†</sup>	
• Cast-in-place in a temporary or permanent casing, or rock	$0.33 f'_c$
• Cast-in-place without a permanent casing	$0.30 f'_c$
• Precast nonprestressed	$0.33 f'_c$
• Precast prestressed	$0.33 f'_c - 0.27 f_{pc}$
b. Nonprestressed reinforcement	
• In compression	$0.4f_y \leq 30,000$ psi
• In tension	$0.5f_y \leq 24,000$ psi
c. Structural steel in compression and tension <sup>‡</sup>	
• H-piles, open-ended pipe piles, and steel casing of concrete-filled pipe piles	$0.35F_y \leq 16,000$ psi

\* Table A.5 is based on Table 1810.3.2.6 of IBC 2012. See IBC 2012, Sec. 1810.3, for conditions where higher allowable stresses are permitted.

<sup>†</sup> Stresses specified apply to the gross cross-sectional area of concrete; the inside area of temporary or permanent casing; and the auger area of uncased piles. The notation  $f'_c$  is the specified compressive strength of concrete or grout tested with cylinders. Strength tests from grout cubes are approximately 25 percent greater than tests from standard cylinders, and should be considered equivalent to 80 percent of the cylinder strength.

<sup>‡</sup> Load-carrying capacity of steel casing is applicable only when the casing thickness is  $\frac{3}{16}$  in. (4.8 mm) or greater.

A.7.4.1.2 *Backfill.* Potential for settlement and heave under applied loads and operating conditions should be considered in the selection of fill material and compaction procedures. The requirements for exterior backfill are for unpaved areas not subjected to load or traffic. Fill material and compaction requirements should be specified for paved areas, roads, and other critical areas.

A.7.4.2.4 *Curing and protection.* See Sec. A.6.4.4 for commentary on curing and protection requirements.

A.7.4.2.5 *Formwork.* Foundation side forms are typically removed the day following concrete placement. Where adequate protection from cold weather is provided, an elapsed time of 12 hours is usually adequate to develop concrete strength suitable for safe form removal.

A.7.4.3 *Piles.* The standard addresses the wide variation in pile types and installation methods by requiring that an installation procedure be developed for the project and that records be kept to confirm construction conforms to the installation procedure. The recommendations of ASCE 20 should be followed in planning and executing a pile installation. Field conditions may require modification of the installation procedure, and those responsible for design and construction should be prepared to address such conditions in a timely manner. Out-of-tolerance piles may require redesign of the pile foundation or pile cap.

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## SECTION A.8: ACCESSORIES

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### Sec. A.8.1 General

A.8.1.2 *Design.* Minimum requirements of this standard are understood to comply with ASCE 7 and OSHA 29 CFR 1910 at the time of writing. ANSI A14.3 provides reference information that can be helpful in understanding and complying with the requirements for the design, construction, and use of fixed ladders, as well as the requirements for cages and ladder safety devices for use with fixed ladders. It is the responsibility of the user of this standard to determine if current building codes and regulations have more restrictive requirements (see foreword, Sec. II.A and Sec. II.B).

A.8.1.3 *Attachment to structure.* Where full welding of attachments to the steel tank is not required, seal welding should be used to minimize corrosion. Bolted connections may be used to attach accessories to brackets welded to the steel tank.

The performance of post-installed anchors in concrete is highly dependent on the type of anchor, quality of concrete, and installation. For critical connections or the attachment of safety-related equipment, only anchors qualified in accordance with appendix D of ACI 318 should be used, and where seismic design is required, the requirements of ASCE 7, Sec. 14.2.2.17 and 14.2.2.18, should be met. For direct tension applications such as hangers, cast-in-place anchors are recommended.

**A.8.1.4 Galvanic and chemical corrosion.** Galvanic corrosion can occur where dissimilar metals are in contact. Isolation or protection should be provided where there is a potential for galvanic corrosion cells to form. For example, unpainted stainless-steel accessories and bolts should be avoided inside the steel tank because of the potential for corrosion.

Aluminum should not be embedded in, or be in contact with, concrete unless it is coated or effectively covered. Aluminum reacts with concrete and can damage the concrete or the aluminum.

## **Sec. A.8.2 Ladders**

**A.8.2.1 General.** Ladders should be welded-steel construction having details that comply with the following:

1. Side rails for carbon steel ladders should be  $\frac{3}{8}$  in. (9.52 mm) by 2 in. (51 mm) minimum, with a clear spacing of 16 in. (400 mm) or greater.
2. Rungs should be  $\frac{3}{4}$  in. (19 mm) or larger spaced at 12 in. (305 mm), and have a skid-resistant surface.
3. Extend ladders 48 in. (1,220 mm) above platforms or landings. Provide ladder extension rails or grab bars at manholes.
4. Provide adequate clearances to obstructions and for climbing.
5. Connect ladders to the adjacent structure by brackets located at intervals not exceeding 10 ft (3.1 m) or at spacing indicated in Table A.6.

Ladders should be of painted or galvanized steel for corrosion protection. The steel tank interior is a relatively severe environment, and heavier ladder components or special coatings may provide additional corrosion protection.

**A.8.2.2 Location.** Interior tank ladders are not a required accessory because of damage that may occur from ice in cold climates (see foreword, Sec. II.F). If interior ladders are installed and ice damage is likely, ladder attachment points should be reinforced to prevent damage to the shell or access tube. Ice damage to ladders is unlikely in areas where the low one-day mean temperature is above 5°F (−15°C).

**A.8.2.3 Fall protection.** The use of safe climbing devices is recommended because they provide a positive means of fall protection. Where specified, they

**Table A.6 Side rail size and spacing (ASTM A36 Steel)**

Side Rail Size, Width × Thickness		Side Rail Spacing	
(in.)	(mm)	(ft)	(m)
2.00 × 0.375	51 × 9.52	7.0	2.1
2.13 × 0.375	54 × 9.52	8.0	2.4
2.25 × 0.375	57 × 9.52	9.0	2.7
2.38 × 0.375	60 × 9.52	10.0 maximum	3.0 maximum
2.00 × 0.50	51 × 12.7	9.5	2.9
2.13 × 0.50	54 × 12.7	10.0 maximum	3.0 maximum

NOTE: Side rail spacing is based on ANSI A14.3 requirements for 2.5-in. × 0.375-in. (64-mm × 9.52-mm) side rails with 10-ft (3.0-m) maximum support spacing and a 250-lb (1.1-kN) load between attachments, or the equivalent strength. This equates to an applied bending stress of 19,200 psi (132 MPa), or approximately  $0.6F_y$  for A36 steel as permitted by AISC. The AISC allowable stress for flexure is the design basis for this table.

should be continuous through intermediate platforms or rest platforms so that personnel are not required to disengage. Otherwise, provide offset ladders with landings (see Sec. A.8.3.4.2 for maximum spacing). Caged ladders should not be used inside the tank.

### Sec. A.8.3 Platforms and Railings

A.8.3.4.2 Intermediate platforms. Current OSHA requirements for the maximum spacing of platforms between offset ladders are

1. 20 ft (6.1 m) for unprotected ladders.
2. 30 ft (9.1 m) for ladders with cages.
3. Unlimited where safe climbing devices are provided.

A.8.3.5 *Roof railing.* OSHA does not mandate the use of a roof railing if other means of fall protection are provided. Toeboards on the roof railing are necessary if other means of providing ground-level protection against falling objects are not provided both during construction and after commissioning. Toeboards, if used, increase the possibility of trapping dirt, water, and ice near the roof openings.

### Sec. A.8.4 Access Openings

A.8.4.1 *Support wall doors.* The number, location, and size of doors should comply with the building code requirements. Doors should be sized for material and equipment required to be moved through the opening. A double personnel door is typically installed where vehicle access is not provided. A separate personnel door should be provided when a vehicle door is furnished.

To minimize the effect of openings on the structural design of the concrete support wall, doors should not be closely spaced or larger than required. Vehicle doors are typically 10-ft (3.1-m) to 12-ft (3.7-m) wide, and may not be practical for small-diameter support walls. Structural design of openings is covered in Sec. 6.3.7.

Doors should have locking devices and windows or louvers should be vandal-resistant to prevent unauthorized access to the support wall interior, access ladder, and equipment.

**A.8.4.3 Access tube.** Antenna cabling, electrical conduit, and piping within the access tube may obstruct ladder access and require a larger diameter than the 42-in. (1,070-mm) minimum. Manhole covers should be constructed from lightweight materials such as aluminum or have a mechanical assist to minimize the effort required for opening and closing.

**A.8.4.4 Tank roof openings.** Roof openings should be large enough for inspection, maintenance, and painting equipment. A removable vent may be used as a roof opening. A tank without an interior ladder presents an additional hazard because of restricted egress.

**A.8.4.5 Tank floor manhole.** A tank floor manhole provides a means of egress other than the roof manholes and provides access for construction and maintenance operations. A 30-in. (760-mm) manhole is a practical minimum size. Larger manhole covers may be difficult to handle.

## **Sec. A.8.5 Permanent Rigging Devices**

Permanent rigging devices are used for tank inspection, maintenance, and painting. Special consideration should be given to their integrity and long-term corrosion protection. The safe loading capacity should be shown on construction drawings and in inspection and maintenance instructions.

## **Sec. A.8.6 Ventilation**

**A.8.6.1 Support structure ventilation.** Natural ventilation of the support structure is provided for air quality and humidity control. Requirements of the standard are for unoccupied space. Additional ventilation requirements may apply when the support structure interior space is used for other purposes. State and local building codes should be consulted to determine the required number, location, and size of vents based on the intended occupancy classification of the structure.

**A.8.6.2 Tank ventilation.** Insect screening has a small mesh that is susceptible to clogging by frost, insects, debris, or paint. In the event of vent failure, a pressure-vacuum relief mechanism is required to prevent damage to the tank.

Reliability of the pressure-vacuum relief mechanism is critical, and it should be checked at frequent intervals for proper operation.

A.8.6.2.2 *Vent.* Water vapor can leave chloride deposits that may cause corrosion of stainless-steel screens.

## Sec. A.8.7 Piping

A.8.7.2 *Steel piping requirements* and

A.8.7.3 *Stainless-steel piping requirements.*

For calculating pipe thickness, allowable hoop tension stress is typically  $0.5F_y$ . To account for permissible pipe thickness underrun of up to 12.5 percent, the allowable stress for nominal thickness is reduced to  $0.44F_y$ .

Local conditions, such as the presence of “aggressive” water—drinking water that can cause corrosion—should be evaluated to ascertain the potential impact on the steel piping and the need for corrosion allowance on the piping.

A.8.7.4 *Inlet/outlet piping.* Separate inlet and outlet lines may be used for tank circulation or other system operating considerations. Inlet/outlet piping should be protected from freezing by insulation, heat-tracing, increasing pipe diameter, circulating water, or a combination of these methods.

A preferred lower support is a structural connection of the base elbow or connecting piping to the support wall or foundation. This detail eliminates differential movement between connecting and riser piping caused by settlement or heave of supporting soils.

A.8.7.5 *Overflow piping.* The overflow prevents overfilling the tank and protects it from overpressure and subsequent damage should the pumps or control valve fail to shut off when the tank is filled to capacity. A properly operated tank should not overflow during normal operation. An overflowing tank is considered an emergency condition, and the malfunction causing the overflow should be determined and corrected as soon as possible. The overflow discharge should be detailed to prevent obstruction by snow or other objects. An air gap at the discharge may be required in some jurisdictions.

A.8.7.6 *Connecting piping.* Connecting piping should have sufficient flexibility to accommodate twice the anticipated differential movement. Mechanical joint sleeves are commonly used for moderate differential movement. Special fittings that provide for rotation and translation may be required where large movements caused by seismic or other events can occur.

### **Sec. A.8.8 Lightning Protection**

NFPA 780 only requires grounding of the steel tank. Air terminals are not required for protection of the steel tank; however, they should be specified if required for obstruction lights, antennas, roof-mounted equipment, and other projecting elements.

### **Sec. A.8.9 Electrical and Lighting**

A.8.9.1 *General.* Electrical and lighting requirements provide for minimum safe access. Electrical service, power distribution, and instrumentation wiring are not addressed in the standard, and should be specified.

A.8.9.2.1 *Interior lights.* Lighting is not usually provided inside the steel tank because of maintenance and safety concerns in the wet environment.

A.8.9.2.4 *Obstruction lighting.* The FAA should be contacted for a determination of lighting and marking requirements. Typically an obstruction light is centrally located on the roof of the tank above all permanent installations.

A typical installation includes a steady burning dual fixture with red globes. The fixture is provided with a lamp-out relay switch and a pilot light located near the electrical panel that indicates when the primary bulb has failed. At some locations, other requirements may apply that include flashing red beacons, white strobes, and tank marking.

### **Sec. A.8.10 Interior Floors**

A.8.10.2 *Slab-on-grade.* Slab-on-grade requirements in the standard are suitable for light truck traffic. Floors intended for parking of heavy vehicles or other heavy loads shall be designed for the specific loading anticipated in accordance with recommendations of ACI 360R.

A.8.10.3 *Intermediate floors.* Floors above grade may be provided for storage or other uses. Connection details of intermediate floors to the support wall may cause significant localized forces that must be considered in design of the support wall. Dead and live loads from structural floors should be included in the design of the support wall and foundation.

### **Sec. A.8.11 Antennas and Communication Equipment**

Details of equipment attachments should have adequate clearance for maintenance.



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## SECTION A.9: INSPECTION AND TESTING

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### Sec. A.9.1 General

A.9.1.1 *Scope.* The standard describes required inspections and tests but does not specifically address implementation, responsibility, and reporting. The job specification must assign responsibility for performing required inspections and tests, preparation of reports, and final disposition of reports, samples, and radiographs.

The scope of inspection and tests in the standard is intended to satisfy the intent of the technical requirements in appendix 11A, Quality Assurance Provisions, of ASCE 7, and chapter 17, Structural Tests and Special Inspections, of the International Building Code. However, the standard does not address administration or implementation requirements.

A.9.1.5 *Test and inspection reports.* The standard requires reporting of test and inspections but does not go into the details: Table 13a lists the sections where specific reporting requirements are given in the standard. Table 13b lists those sections where the specifier has to define the level of reporting of tests and inspections that will be required.

### Sec. A.9.2 Geotechnical and Foundation

A.9.2.1 *Shallow foundations.* It is not uncommon for soils to vary between borings, and inspection by the geotechnical engineer of the bearing stratum of shallow foundations provides the opportunity to confirm that the exposed bearing stratum is the material encountered in the soils investigation.

A.9.2.2 *Deep foundations.* Specific requirements for inspection of deep foundations are beyond the scope of the standard. The following are some references that may be consulted in developing job specifications for a project.

1. General:

- ASCE 7, appendix 11A, Quality Assurance Provisions. Defines minimum inspection criteria, quality assurance, and responsibility for inspection.
- ASCE 20, Standard Guidelines for the Design and Installation of Pile Foundations.

2. Driven piles:
  - FHWA,\* HI 97-013, *Design and Construction of Driven Pile Foundations*, Volume 1.
3. Drilled piers or piles:
  - ACI 336.1, Specification for the Construction of Drilled Piers.
  - ACI 336.3, Report on Design and Construction of Drilled Piers.
  - Deep Foundation Institute, *Auger Cast-in-Place Pile Model Specification*.
  - Deep Foundation Institute, *Inspector's Guide to Augered Cast-in-Place Piles*.
  - FHWA, IF-99-025, *Drilled Shafts: Construction Procedures and Design Methods*.

### Sec. A.9.3 Concrete

The testing and inspection requirements are based on Section 1 of ACI 301. Field tests of concrete required in Sec. 9.3.3 of this standard should be made by an ACI Field Testing Technician Grade 1 in accordance with ACI publication ACI CP1 or equivalent. Equivalent certification programs should include requirements for written and performance examinations as stipulated in ACI CP1.

The following additional concrete inspection and testing may be performed when specified:

- Inspect concrete batching, mixing, and delivery operations.
- Sample and test materials at plants or stockpiles.
- Sample concrete at point of placement or other locations specified.
- Review manufacturer's report for each shipment of cement and reinforcing steel, and conduct laboratory tests or spot checks of the materials received for compliance with specifications.

### Sec. A.9.4 Steel Tank: Materials and Tolerances

A.9.4.1 *Materials.* Reporting requirements for ASTM materials included in the standard are covered by either ASTM A6 or ASTM A20. The general requirements for reporting may be applied to material covered by CSA G40.21.

### Sec. A.9.5 Steel Tank: Welding

#### A.9.5.1 *General.*

A.9.5.1.1 *Time of inspection.* It is recommend that welding inspection be performed as early as possible as the work progresses in order to identify prob-

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\* Federal Highway Administration, 1200 New Jersey Avenue, SE, Washington, DC 20590.

lems with individual welders or processes. An equal number of locations should be inspected for each welder, unless there is a significant imbalance with regard to the amount of welding performed by individual welders.

#### A.9.5.2 *Inspection requirements.*

A.9.5.2.2 Shell plate joints. Inspection of complete joint penetration butt welds in shell plate is by spot radiographic examination, and the method is recognized as an effective inspection tool. It is also an aid to quality control when testing is performed directly after completion of a unit of weld, and corrective steps are taken to remedy any unsatisfactory work to improve weld quality. Spot radiographic examination in accordance with this standard will not ensure a predetermined level of quality throughout the entire structure, and a structure meeting the requirements of this standard may still contain defects that might be disclosed by further testing.

Radiographic testing is used for inspection of all shell plate butt joints with complete joint penetration welds and joints that are categorized as primary or secondary stress joints. Primary stress joints are defined as “welded butt joints subject to transverse tension stress resulting from the weight or pressure of the contained water.” An example of primary tension stress is the hoop stress acting on the vertical weld seams in a water-filled cylinder. An example of primary compression stress is the meridional stress in water-filled cone acting on the circumferential weld seams. Secondary stress joints are welded butt joints other than primary stress joints that are either in compression or are subject to small tension stress. Generally, they are welds where partial joint penetration welds would be adequate. The circumferential weld seams of cylindrical sections and cone sections will generally be classified as secondary stress joints.

Inspection by radiographic methods is not intended to apply to roof or roof-to-shell joints not subject to the weight or pressure of the stored water, welds connecting accessories to the tank, or fillet welds.

A.9.5.3 *Visual inspection.* Requirements for visual inspection are based on AWS D1.1, Sec. 6.9. Except for limits on undercut and maximum reinforcement of shell butt joints, acceptable weld profiles are required to conform to AWS D1.1, Sec. 5.24, which includes Figure 5.4, illustrating acceptable and unacceptable profiles that include the following:

- Face reinforcement of groove welds. Maximum is  $\frac{1}{8}$ -in. (3.18-mm) height, except Table 16 of this standard applies to shell plate butt joints.

**Table A.7 Fillet weld convexity**

Width of Weld Face or Individual Surface Bead, $W$		Maximum Convexity	
(in.)	(mm)	(in.)	(mm)
$W \leq \frac{5}{16}$	$W \leq 7.94$	$\frac{1}{16}$	1.59
$\frac{5}{16} < W < 1$	$7.94 < W < 25$	$\frac{1}{8}$	3.18
$W \geq 1$	$W \geq 25$	$\frac{3}{16}$	4.76

- Overlap resulting from the protrusion of weld metal beyond the weld toe or weld root. None is permitted.
- Undercut. Maximum undercut for shell plate butt joints is per Sec. 5.4.4.2 (item 3) of this standard. For other welds, undercut is limited to the following:
  - a.  $\frac{1}{32}$  in. (0.80 mm) for material less than 1 in. (25 mm), except a maximum of  $\frac{1}{16}$  in. (1.59 mm) is permitted for an accumulated length of 2 in. (50 mm) in any 12-in. (300-mm) length.
  - b.  $\frac{1}{16}$  in. (1.59 mm) for any length of weld for material 1-in. (25-mm) thickness or greater.
- Insufficient (undersized) throat or fillet weld leg size. None is permitted.
- Convexity of fillet welds. Not to exceed limits of Table A.7.

#### A.9.5.4 Radiographic examination.

A.9.5.4.1 (item 3) Image quality indicators (IQIs). Table A.8 is included for reference and contains the IQI selection criteria in Table T-276 of ASME Boiler and Pressure Vessel Code, Section V, Article 2.

A.9.5.4.1 (item 4) Evaluation. Geometric unsharpness is for guidance only, and is defined in clause T-274 of ASME Boiler and Pressure Vessel Code, Section V, Article 2. The method of determining geometric unsharpness is in accordance with *Standard Guide for Radiographic Testing SE-94*. Limits for geometric unsharpness per clause T-285 of Section V, Article 2, are included here for reference:

- 0.20 in. (5.08 mm) for plate under 2-in. (51-mm) thickness.
- 0.30 in. (7.62 mm) for plate under 2-in. through 3-in. (51-mm through 76-mm) thickness.

A.9.5.4.2 Definitions for radiographic evaluation. These definitions are in general agreement with AWS A3.0, Standard Welding Terms and Definitions.

**Table A.8 IQI selection**

Plate Thickness Range		Hole-Type IQI Designation		Wire-Type IQI Essential Wire	
(in.)	(mm)	Source Side	Film Side	Source Side	Film Side
Up to 0.25 inclusive	Up to 6.35 inclusive	12	10	5	4
0.25 to 0.375 inclusive	6.35 to 9.52 inclusive	15	12	6	5
0.375 to 0.50 inclusive	9.52 to 12.7 inclusive	17	15	7	6
0.50 to 0.75 inclusive	12.7 to 19 inclusive	20	17	8	7
0.75 to 1.00 inclusive	19 to 25 inclusive	25	20	9	8
1.00 to 1.50 inclusive	25 to 38 inclusive	30	25	10	9
1.50 to 2.00 inclusive	38 to 51 inclusive	35	30	11	10
2.00 to 2.50 inclusive	51 to 64 inclusive	40	35	12	11
2.50 to 4.00 inclusive	64 to 100 inclusive	50	40	13	12

Source: Data from Table T-276, ASME Boiler and Pressure Vessel Code, Section V, Article 2.

ASME Boiler and Pressure Vessel Code, Section VIII, Division I, appendix 4, Rounded Indications Charts Acceptance Standards for Radiographically Determined Rounded Indications in Welds, is referenced in the standard to evaluate rounded discontinuities. The definition for the term *aligned discontinuities (elongated)* in the standard is not the same as that in appendix 4 where the term *aligned indications* is used and defined as:

“Aligned indications: a sequence of four or more rounded indications shall be considered aligned when they touch a line parallel to the length of the weld drawn through the center of the two outer rounded indications.”

A.9.5.4.3 Acceptance criteria. The acceptance criteria used in the standard are based on the requirements of ASME Boiler and Pressure Vessel Code, Section VIII Division 1, clause UW-52, Spot Examination of Welded Joints, except for the following:

- Consideration of incomplete fusion or penetration discontinuities in secondary stress weldments as separate from cracks. The exception is taken because clause UW-52 is intended for primary stress weldments, and it and other ASME requirements do not directly deal with acceptance criteria for the secondary stress weldments covered by the standard.
- Consideration of rounded discontinuities. Clause UW-52 states that “rounded indications are not a factor in the acceptability of welds not

**Table A.9 Maximum acceptable rounded discontinuities for RT\***

Weld Size, E	Acceptable Rounded Discontinuities		Maximum Nonrelevant
	Random	Isolated	
inch units:			
$\frac{3}{16}$ to $\frac{1}{4}$ inclusive	$E/4$ , $\frac{5}{32}$ maximum	$E/3$ , $\frac{1}{4}$ maximum	$\frac{1}{64}$
Over $\frac{1}{4}$ to 2 inclusive	$E/4$ , $\frac{5}{32}$ maximum	$E/3$ , $\frac{1}{4}$ maximum	$\frac{1}{32}$
Over 2	$\frac{5}{32}$ maximum	$\frac{3}{8}$ maximum	$\frac{1}{16}$
mm units:			
4.76 to 6.35 inclusive	$E/4$ , 3.97 maximum	$E/3$ , 6.35 maximum	0.40
Over 6.35 to 51 inclusive	$E/4$ , 3.97 maximum	$E/3$ , 6.35 maximum	0.80
Over 51	3.97 maximum	9.52 maximum	1.59

\* Based on example Table 4-1 of ASME Boiler and Pressure Vessel Code, Section VIII, Division I, appendix 4.

required to be fully radiographed.” The consideration of rounded discontinuities is based on Clause UW-51, Radiographic and Radioscopic Examination of Welded Joints.

A.9.5.4.3 (item 5) Acceptance criteria for rounded discontinuities. The following is included for reference and describes acceptance criteria for rounded discontinuities found in ASME Boiler and Pressure Vessel Code, Section VIII, Division I, appendix 4. Appendix 4 uses the term *indications* instead of *discontinuities*, and the terms can be considered interchangeable for purposes of this discussion.

Maximum permissible size of random and isolated discontinuities is shown in Table A.9.

Aligned discontinuities (rounded) are acceptable when the summation of the diameters of the discontinuities is less than weld size  $E$  in a length of  $12E$ . Appendix 4 defines *aligned indications* (discontinuities) as:

“A sequence of four or more rounded indications shall be considered aligned when they touch a line parallel to the length of the weld drawn through the center of the two outer rounded indications.”

Appendix 4 includes rounded indication charts that are used to evaluate acceptance or rejection of radiographs. The illustration for clustered indications shows up to four times as many indications as for random indications. The length of an acceptable cluster is not to exceed the lesser of 1 in. or  $2E$ . Where more than one cluster is present, the sum of the lengths of the clusters is not to exceed 1 in. (25 mm) in any 6 in. (150 mm) of length.

Clearances between discontinuities shown on the rounded indication charts are as follows:

- Minimum of  $3L_g$  between groups of aligned rounded discontinuities, where  $L_g$  is the length of the longest adjacent group being evaluated.
- Minimum of 1 in. (25 mm) between an isolated rounded discontinuity of maximum permissible size and any adjacent discontinuity.

A.9.5.6 *Pressure/vacuum testing.* ASME Boiler and Pressure Vessel Code, Section V, Article 10, describes bubble testing by vacuum box and direct pressure. Mandatory appendix I covers the direct pressure technique, and mandatory appendix II covers the vacuum box technique. A partial vacuum of 2 psi (0.0138 MPa) below atmospheric pressure is required for vacuum testing, but a minimum pressure for direct pressure testing is not specified. The direct pressure should not exceed the safe working load or structural capacity of anchorage devices and the connecting welds of the components being pressurized.

#### Cited References

The following documents are cited in this appendix only.

#### Section A.4

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#### **Section A.5**

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#### **Section A.6**

A.6-1. American Concrete Institute, 334.2R-91, Reinforced Concrete Cooling Tower Shells—Practice and Commentary.



## APPENDIX B

### Information Provided by Purchaser

*This appendix is for information only and is not part of ANSI/AWWA D107.*

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#### SECTION B.1: REQUIRED INFORMATION

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The following information is required when ANSI/AWWA D107 is used to specify a composite elevated tank:

1. Project location.
2. Tank capacity.
3. Maximum head range.
4. Elevation of the TCL and BCL.
5. Elevation of finished grade.
6. Maximum fill rate.
7. Size of inlet/outlet pipe.

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#### SECTION B.2: PURCHASER OPTIONS

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ANSI/AWWA D107 contains optional requirements that the purchaser may incorporate into the project documents. Table B.1 is a comprehensive list of these options and the corresponding defaults.

**Table B.1 Optional requirements and defaults**

Section	Option	Default
II.A	Specify if building codes apply and to what extent.	Building codes not considered.
II.C	Specify loads for service not covered by this standard.	No special loads considered.
II.D	Specify if professional engineer (PE) certification is required.	PE certification not required.
1.4.3	Specify if operating and maintenance instructions are to be provided.	Instructions not provided.

*(continued next page)*

**Table B.1 Optional requirements and defaults (continued)**

Section	Option	Default
4.2.1	Specify minimum loads different than the standard.	Loads required by the standard.
4.2.4	Specify live loads if different than ASCE 7.	ASCE 7 live loads.
4.2.5.3	Specify flat roof snow load if different than the standard.	Snow load based on ASCE 7.
4.2.6.1	Specify wind loads if different than the standard.	Wind loads based on ASCE 7 and 30 psf (1,440 Pa) minimum velocity pressure.
4.2.6.3	Specify wind exposure category.	Wind exposure category C.
4.2.6.4	Specify if shrouding is required.	Shroud loads and wind loads on shroud not considered.
4.2.7.1	Specify seismic loads if different than ASCE 7.	Seismic loads based on ASCE 7.
4.2.9.3	Specify if structure is to be designed for future construction loads (e.g., intermediate floors).	Future construction loads not considered.
5.2.1	Specify materials not in the standard that are permitted.	Materials to comply with requirements of the standard.
5.3.2.2	Specify minimum plate thickness different than the standard.	No exceptions taken.
5.3.2.3	Specify corrosion allowance. If corrosion allowance is required, specify which elements receive corrosion allowance.	Zero corrosion allowance.
5.3.5.5 (item 2)	Specify if seal welding of exposed weld joints on tank exterior and interior is required; usually roof interior.	Seal weld exposed joints.
5.4.5	Specify painting system/testing sequence.	See Sec. A.5.4.5 for hydrotest sequence discussion.
6.4.2.1 (item 3)	Specify exposure for air entrainment.	Moderate exposure as defined in ACI 318.
6.4.7.1	Specify if additional finishing is required for formed surfaces.	No additional finishing.
6.4.7.2	Specify finish of unformed surfaces when other than floated finish.	Floated finish.
7.2.2	Specify pile types and materials other than permitted in ASCE 20.	Pile types and materials per ASCE 20.
7.3.1.1	Specify increase in allowable bearing capacity and allowable pile capacity permitted for load combinations that include wind or seismic loads.	A one-third increase in allowable bearing capacity and allowable pile capacity is permitted.

*(continued next page)*

**Table B.1 Optional requirements and defaults (continued)**

Section	Option	Default
7.3.3.2	Specify pile design requirements.	Pile design as specified in Sec. 7.3.3.2.
7.3.4.1	Specify minimum foundation depth.	Minimum foundation depth per Figure 1.
7.4.1.2 (item 1)	Specify compaction requirement for backfill inside support wall.	95 percent standard Proctor density.
7.4.1.2 (item 2)	Specify material to be used for backfill outside the support wall.	Unclassified soils may be used compacted to 90 percent standard Proctor density.
7.4.2.2.1 (item 3)	Specify exposure for air entrainment.	Moderate exposure as defined in ACI 318.
7.4.2.7.2	Specify finish of unformed surfaces when other than floated finish.	Floated finish.
7.4.3.5	Specify pile tolerances other than those in the standard.	Tabulated values.
8.1.5	Specify other coating systems for steel.	Zinc coating and pint systems.
8.2.2 (item 4)	Specify if tank roof to tank floor ladder is required.	Not required.
8.2.3	Specify ladder fall protection.	Safe climbing devices on all ladders where required by OSHA.
8.3.1	Specify platform and railing materials other than steel.	Painted or galvanized steel.
8.3.4.2	Specify if intermediate platforms are required.	Only required for offset ladders and piping access.
8.3.5	Specify if a roof guardrail is required.	Roof guardrail is not required.
8.4.1	Specify if vehicle door is required.	Vehicle door not required.
8.4.1.1	Specify personnel door requirements.	Personnel door requirements per Sec. 8.4.1.1.
8.4.4	Specify number of roof openings.	Minimum of two 30-in. (760-mm) roof openings.
8.5	Specify permanent rigging devices.	Permanent rigging devices as specified in Sec. 8.5.
8.6.1	Specify number and size of support structure vents.	One vent with opening of 500 in. <sup>2</sup> (0.322 m <sup>2</sup> ).
8.6.2.2	Specify vent screen.	Aluminum, brass, or fiberglass insect screen.
8.7.1	Specify material of riser piping located outside the stored water area.	Steel piping per Sec. 8.7.2 or stainless-steel piping per Sec. 8.7.3.
8.7.2	Specify steel piping requirements.	Steel piping requirements per Sec. 8.7.2.
8.7.2 (item 4)	Specify if inlet/outlet pipe is to be lined, and if so, the type of lining.	Lining per AWWA standards listed in Sec. 8.7.2 (item 4).
8.7.3	Specify stainless-steel piping requirements.	Stainless-steel piping requirement per Sec. 8.7.3.

*(continued next page)*

**Table B.1 Optional requirements and defaults (continued)**

Section	Option	Default
8.7.4 (item 1)	Specify configuration of inlet/outlet piping and size.	Single inlet/outlet pipe.
8.7.4 (item 2)	Specify if removable silt stop is required.	Not required.
8.7.5 (item 1)	Specify minimum overflow pipe diameter.	6-in. (150-mm) minimum diameter.
8.7.5 (item 2)	Specify configuration of overflow piping.	Overflow pipe may be attached to interior or exterior of access tube.
8.7.5 (item 5)	Specify configuration of overflow discharge.	Discharge onto concrete splash pad at grade.
8.7.6	Specify piping that connects to riser piping.	
8.7.7	Specify minimum pipe cover.	Minimum pipe cover per Figure 2.
8.9.2	Specify electrical equipment.	Electrical equipment as specified in Sec. 8.9.2.
8.9.2.4	Specify if obstruction lighting is required.	Not required.
8.9.3	Specify wiring materials.	Wiring materials as specified in Sec. 8.9.3.
8.10.2	Specify if slab-on-grade is required.	Slab-on-grade required.
8.10.2 (item 4)	Specify if slab-on-grade floor drains are required.	Not required.
8.10.3	Specify if intermediate floors are required.	Not required.
8.10.3 (item 3)	If intermediate floors are required, specify live load.	100 psf (4,790 Pa).
8.10.3 (item 4)	Specify concrete floor finish.	Troweled finish.
8.11 (item 1)	Specify antenna and communications equipment loads.	
9.1.4.1 (item 3)	Specify certification requirements for welding inspectors.	Certification complying with Sec. 9.4.1 (item 1) and 9.1.4.1 (item 2)
9.1.5	Specify extent to which tests listed in Tables 13a and 13b are required.	Not required unless mandated by standard.
9.4.1	Specify if material test reports for the steel tank are required.	Not required.
9.5.1.2	Specify when a report certifying their work has been inspected in accordance with Section 9 is required.	Not required.
9.7.1	Specify when a leak test is not required.	Leak test is required.
9.7.2	Specify when settlement monitoring during hydrotest is required.	Not required.

# APPENDIX C

## Recommendations for Geotechnical Investigation

*This appendix is for information only and is not part of ANSI/AWWA D107.*

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### SECTION C.1: GENERAL

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**C.1.1 Scope.** This appendix is a guideline for obtaining a geotechnical investigation for composite elevated tanks. The geotechnical investigation includes field investigation, laboratory testing, engineering analysis, and recommendations.

**C.1.2 Purpose.** The purpose of the geotechnical investigation is to obtain information on the subsurface conditions at the site for design and construction of foundations.

**C.1.3 Procurement.** The geotechnical investigation is usually procured by the purchaser during planning of a project, and the report is typically included with the project documents. A geotechnical engineer familiar with the geological conditions at the site should be retained to provide the geotechnical investigation and report. The geotechnical engineer should also be involved during final foundation design and construction in order to confirm recommendations and provide further testing and inspection as required. Such involvement allows the geotechnical engineer to be aware of unusual or unforeseen conditions that may affect the recommendations or performance of the foundation.

**C.1.4 Geotechnical engineer responsibilities.** A geotechnical engineer familiar with the design criteria should be responsible for developing and coordinating the investigation and testing program. The geotechnical engineer should consider imposed loads, topography, subsurface conditions, and soil and rock properties to provide design, construction, and inspection recommendations.

The geotechnical engineer should be provided with preliminary structure and load information including anticipated height, support wall diameter, dead load and water weights, and shear and moment at grade caused by wind and seismic loads.

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## SECTION C.2: DESIGN CRITERIA

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**C.2.1 Strength.** A factor of safety of 3.0 or greater is required for bearing capacity of shallow foundations. The factor of safety for pile foundations should comply with Table C.1. The allowable bearing capacity or pile capacity is normally increased by one-third for load combinations including wind or seismic loads.

**C.2.2 Settlement guidelines.** The combined foundation and concrete support wall form a rigid structure that will experience negligible out-of-plane settlement. Subsurface deformations that require consideration are total settlement and differential settlement that causes tilting of the structure. The following settlement limits are typically used for composite elevated tanks:

1. Total settlement for shallow foundations: 3 in. (75 mm).
2. Total settlement for deep foundations:  $\frac{3}{4}$  in. (20 mm).
3. Tilting of the structure:  $\frac{1}{800}$ . Larger differential tilt is permitted when the effects of tilting are included in the structural design. Maximum tilt should not exceed  $\frac{1}{300}$ .

At some sites, it may not be feasible to provide a foundation that meets the above settlement limits. Larger settlement may be permitted; however, it is important to design for the effects of such deformations on the structure, connecting piping, or other accessories.

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## SECTION C.3: FIELD INVESTIGATION

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**C.3.1 General.** Field investigation should include sufficient sampling, testing, and observations to determine site conditions and engineering properties of the supporting soils for design and construction of the foundation.

**Table C.1 Minimum factor of safety for pile foundations**

Method of Establishing Ultimate Pile Capacity	Minimum Factor of Safety
Analysis using engineering principles	3.0
High-strain dynamic testing of driven piles in accordance with ASTM D4945	2.25
Static load testing in accordance with ASTM D1143, and high-strain dynamic testing of driven piles in accordance with ASTM D4945 using signal matching analysis or other in-situ load tests that determine end bearing, side friction, or both	2.0

C.3.2 *Borings.* At least three equally spaced borings located on the anticipated diameter of the foundation should be provided. Borings should extend sufficient depth to identify strata that may be significantly affected by imposed foundation loads. It may be necessary to change the number and depth of borings if unanticipated subsurface conditions are encountered.

C.3.3 *Sampling and field testing.* Sampling should generally be performed at 2.5-ft (0.8-m) intervals for the first 10 to 15 ft (3.0 to 4.6 m), and at 5-ft (1.5-m) intervals thereafter. Split-spoon sampling per ASTM D1586 is generally used for granular soils, and Shelby tube samplers per ASTM D1587 are generally used for cohesive soils. Field testing with pocket penetrometer, shear vane, cone penetration, and pressure meter may be used to determine physical properties of subsurface soils.

C.3.4 *Groundwater.* Boreholes should be bailed and water levels measured to determine groundwater conditions.

C.3.5 *Completion.* Boreholes should be filled and plugged in accordance with applicable regulations.

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## SECTION C.4: LABORATORY TESTING

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C.4.1 *General.* Laboratory testing on representative samples should be conducted to determine physical characteristics of subsurface soils. Testing should be in accordance with applicable ASTM standards.

C.4.2 *Classification.* Soils should be classified in accordance with the Unified Classification system. Deleterious materials, chemically active materials, or other special soil conditions should be noted.

C.4.3 *Strength.* Shear strength and angle of internal friction for bearing capacity should be determined by testing or by indirect methods. Potential strength change with moisture content variation should be considered.

C.4.4 *Settlement.* Testing to evaluate deformation characteristics of soils should be conducted. Tests may include consolidation testing per ASTM D2435.

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## SECTION C.5: ENGINEERING ANALYSIS AND RECOMMENDATIONS

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**C.5.1 General.** Shallow and deep foundation systems should be evaluated based on the results of field and laboratory testing, structural loads, and settlement limitations. Recommendations for design and construction and predicted settlements should be provided.

**C.5.2 Shallow foundations.** Ultimate bearing capacity and net allowable bearing capacity should be determined based on the design criteria in Section C.2. The effect of short duration loads such as wind or seismic should be considered in evaluating allowable bearing capacity. Resistance to lateral loads by friction, adhesion, or passive resistance should be defined.

**C.5.3 Deep foundations.** Ultimate axial compression and tension capacity and allowable pile capacities should be determined based on the design criteria in Section C.2. The effect of short-duration loads such as wind or seismic should be considered in evaluating the allowable pile capacity. Soil resistance parameters for lateral loads should be defined. Group effects for axial and lateral loads should be defined, and the spacing of individual piles should be established. The effects of downdrag or expansive soils should be included in evaluating pile capacity.

**C.5.4 Other considerations.** The following should be included where applicable:

1. Classification of the soil type for determining site amplification factors.
2. Presence of permanent high water table for stability calculations.
3. Construction dewatering requirements.
4. Recommendations for excavation and backfilling.

**C.5.5 Report.** A report that documents the results of field investigation, laboratory testing, analysis, and recommendations should be prepared.

### References

The following standards, specifications, or technical publications are referenced in this appendix. Other references are listed in Section 2.

ASTM D1143—Standard Test Methods for Deep Foundations Under Static Axial Compressive Load.

ASTM D1586—Standard Test Methods for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.



ASTM D1587/D1587M—Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.

ASTM D2435/D2435M—Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.



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