ANSI/AWWA **D103-19**

(Revision of ANSI/AWWA D103-09)

AWWA Standard

Factory-Coated Bolted Carbon Steel Tanks for Water Storage

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American Water Works Association

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AWWA Standard

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Foreword

This foreword is for information only and is not a part of ANSI/AWWA D103.*

I. Introduction.

I.A. *Background.* This standard covers factory-coated bolted carbon steel tanks for water storage and is based on the accumulated knowledge and experience of manufacturers of bolted carbon steel tanks.[†]

I.B. *History.* The first version of this standard was prepared in cooperation with the Bolted Tank Manufacturer's Association and was issued in 1980. It was prepared in response to the increasing use of bolted tanks for water storage. ANSI/AWWA D103-80 was later updated and approved as ANSI/AWWA D103-87 on June 14, 1987.

Subsequent editions of ANSI/AWWA D103 were approved by the AWWA Board of Directors on June 15, 1997, and Jan. 25, 2009. This edition was approved on June 7, 2019.

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). 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.[‡] 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:

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.

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

† The word *tanks* is used hereinafter broadly in place of the lengthy phrase *standpipes or reservoirs for water storage*.

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

§ NSF International, 789 North Dixboro Road, Ann Arbor, MI 48113.

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3. Other references, including AWWA standards, *Food Chemicals Codex, Water Chemicals Codex,*[*] 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 jurisdiction. 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 D103 does not address additives requirements. Thus, 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. *Purchase.* When tanks are purchased using this standard, the purchaser must specify certain basic requirements. The purchaser may desire to modify, delete, or amplify sections of this standard to suit special conditions. It is strongly recommended that such modifications, deletions, or amplifications be made by supplementing this standard rather than by rewriting or incorporating sections from this standard into a separate specification.

II.B. *Design and Construction.* The details of design and construction covered by this standard are minimum requirements. A tank cannot be represented as adhering to the provisions of ANSI/AWWA D103 if it does not meet the minimum requirements of this standard.

II.C. *Coatings.* Tanks covered by this standard shall be supplied with factory-applied coatings. Field coating is limited to repair of damaged coatings.

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

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Tanks with factory-applied coatings and bolted construction have a long life expectancy. Regular inspection and repair of damaged or deteriorated areas may be the determining factors in the length of tank life.

II.D. *Foundations.* Tank foundations are one of the more important aspects of tank design. Detailed requirements for tank foundations are covered in Section 13 of this standard. This standard does not require the manufacturer or constructor to be responsible for the design of the tank foundation unless specified. An adequate soil investigation at the tank site, including recommendations of the type of foundation to be used, the depth of foundation required, specification and compaction of sub-base materials, and the design soil-bearing pressure, should be obtained. This information, as well as specifications for an adequate soil investigation, should be established by a qualified geotechnical engineer. A drainage inlet structure or suitable erosion protection should be provided to receive the discharge from the tank overflow. The overflow should not be connected directly to a sewer or a storm drain without an air break.

II.E. *Annual Inspection, Maintenance, and Operation.* Annual inspection and maintenance is important if maximum tank life is to be attained. Complete interior and exterior inspections should occur at least every three to five years. In particular, accumulations of dirt and weeds from around the outside base of the tank, which may trap moisture and accelerate corrosion, as well as accumulated silt inside on the floor, should be removed. Refer to AWWA Manual M42, *Steel Water-Storage Tanks*, for guidance concerning inspection and maintenance.

Sufficient water replacement and circulation is necessary to prevent freezing in the tank and piping. Where low usage may result in the possibility of freezing, the water may need to be wasted or heat provided to prevent freezing. The purchaser is referred to NFPA* 22, Standard for Water Tanks for Private Fire Protection, for heater sizing. Purchasers are cautioned against allowing ice to build up for use as insulation because the ice may break loose and damage the tank.

II.F. *Disinfection Procedures.* This standard does not cover disinfecting procedures† (see Sec. 11.3). If the disinfecting is to be performed by the tank constructor, the purchaser must specify the disinfecting procedure to be used (see ANSI/AWWA C652).

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

† Various disinfection procedures are presented in ANSI/AWWA C652, Disinfection of Water-Storage Facilities.

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III. Use of This Standard. It is the responsibility of the user of an AWWA standard to determine that the products described in that standard are suitable for use in the particular application being considered.

III.A. *Purchaser Options and Alternatives.* The following information should be provided by the purchaser.

1. Standard used—that is ANSI/AWWA D103, Factory-Coated Bolted Carbon Steel Tanks for Water Storage, of latest revision.
2. Whether compliance with NSF/ANSI 61, Drinking Water System Components—Health Effects, is required.
3. Details of federal, state, and local requirements (Sec. 1.1.3).
4. Tank geometry.
    a. Dimensions, capacity, and maximum water level (MWL) above top of foundation.
    b. The bottom capacity level (BCL) of the tank when empty if different from the level when the tank would be emptied through the specified discharge piping.
    c. Type of roof. Steel roof or aluminum dome.
    d. Type of bottom. Steel tank bottom or concrete bottom with embedded steel base setting ring (Sec. 13.4.6).
    e. Vertical distance from finished ground level to the crown of inlet and outlet pipes (pipe cover) at tank foundation (Sec. 13.7.2).
5. Tank accessories and requirements.
    a. The type of pipe and fittings for fluid conductors and the type of pipe joint if different from that permitted in Sec. 4.9.
    b. Locations of manholes, ladders, and accessories (Section 7). NOTE: Only one shell manhole will be provided unless the purchaser otherwise specifies (Sec. 7.1).
    c. The number and location of pipe connections, and type and size of pipe to be accommodated.
    d. If a removable silt stop is required (Sec. 7.2.1).
    e. Overflow type, whether stub or to ground; size of pipe; pumping and discharge rates (Sec. 7.3).
    f. If the exterior ladder or access to roof hatches and vents is to be omitted (Sec. 7.4).
    g. If ladder cages, rest platforms, ladder locks, roof-ladder handrails, or other safety devices in excess of OSHA are required (Sec. 7.5).

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h. If a special screen is required for tank vent (Sec. 7.7).
i. If shop inspection is required, and whether certified mill-test reports are required (Sec. 11.1).
j. If a cathodic protection system is desired, specify the type of system (e.g., an automatically controlled impressed current cathodic protection system or a sacrificial anode cathodic protection system).

6. Tank design criteria.
   a. Snow load. If snow loading is to be reduced (Sec. 5.2.4).
   b. Wind load. Specific wind-load requirements, including whether a sliding check for self-anchored tanks is required (Sec. 5.2.6 and Sec. 15.3).
   c. Seismic load. Risk Category for the tank, or if the seismic design is to be omitted (Sec. 5.2.1 and Sec. 14.2.1).
   d. If seismic design of roof framing and columns is required and the amount of live loads to be used (Sec. 14.3.4.4).
   e. Whether the site-specific seismic design procedure is required (Sec. 14.2.7).
   f. Other loads. Equipment, platforms, walkways, and all additional loads including live, dead, static, and dynamic loads.
7. Foundation.
   a. Geotechnical report. Summary of soil investigation, including foundation-design criteria (Sec. 13.2), type of foundation (Sec. 13.4), depth of foundation below existing grade, site class for seismic areas, and design soil-bearing pressure, including appropriate factor of safety.
   b. When a pile-supported foundation is required, the pile type and the depth below existing grade (Sec. 13.5.4).
   c. The elevation of concrete foundations above the finish grade (Sec. 13.5.1).
8. Field construction.
   a. The prepared site location.
   b. Type of road available for access to the site and whether the road is public or private.
   c. Name of and distance to the nearest town.
   d. Name of and distance to the nearest railroad siding.
   e. Availability of electric power; who furnishes it and at what fee, if any; what voltage is available; whether direct or alternating current; and, if alternating current, what cycle and phase.

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f. Availability of compressed air and what pressure, volume, and fee are available, if any.
g. Desired time for completion.
h. Any materials furnished by the purchaser to be used by the constructor for construction of the tank.
i. Necessary blinding fittings for leak testing.

III.B. *Information to Be Provided With the Bid.*

1. Detailed Submittals.
   a. Detailed drawing of the tank with dimensions, including the diameter, shell height, height to the maximum water level (MWL), bottom, roof, coatings, details of bolted joints, accessories, ladders, and type and size of plates, members, and anchorage, as applicable.
   b. Loads imposed on the foundation, including moment and shear under seismic and wind loading.

III.C. *Modification to Standard.* Any modification to the provisions, definitions, or terminology in this standard must be provided by the purchaser.

IV. Major Revisions. Major revisions made to the standard in this edition include the following:

1. Sec. 5 was revised to include risk category definitions from ASCE 7-10. Snow loads were revised to comply with ASCE 7-10.

2. Sec. 14 was revised so that seismic loading complies with ASCE 7-10. The site classification procedure was deleted and a reference to the procedure in ASCE 7-10 was added. Spectral response acceleration maps and the long-period transition period map were deleted and references to the maps in ASCE 7-10 were added. The vertical design acceleration was revised to include values for responses limited by buckling and responses not limited by buckling. Requirements were added for situations when the required freeboard is not provided. The slosh height equation was revised to match ASCE 7-10. The equation for allowable lateral shear for flat-bottom tanks was revised to show a service-level shear resistance similar to API 650 and ASCE 7-10.

3. Sec. 15 was revised so that wind loads complies with ASCE 7-10. The basic wind speed map was deleted and a reference to the basic wind speed map in ASCE 7-10 was added.

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 e-mail at standards@awwa.org.

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(Revision of ANSI/AWWA D103-09)

American Water Works Association

Dedicated to the World's Most Important Resource®

AWWA Standard

Factory-Coated Bolted Carbon Steel Tanks for Water Storage

SECTION 1: GENERAL

Sec. 1.1 Scope

The purpose of this standard is to provide minimum requirements for the design, construction, inspection, and testing of new cylindrical, factory-coated, bolted carbon steel tanks for the storage of water. This standard is only applicable to tanks with a base elevation substantially at ground level.

1.1.1 *Tank roofs.* All tanks storing potable water shall have roofs. Roofs may be column-supported, self-supported, or aluminum dome. Tanks storing nonpotable water may be constructed without roofs.

1.1.2 *Items not described.* This standard does not cover all details of design and construction. Details that are not addressed shall be designed and constructed to be as adequate and as safe as those that would otherwise be provided under this standard. This standard is not applicable to tanks of corrugated construction. This standard is not applicable to tanks constructed of stacked plates or sheets laminated to form multiple layers.

1.1.3 *Local requirements.* This standard is not intended to cover storage tanks erected in areas subject to regulations more stringent than the requirements contained within this standard. In such cases, this standard should be followed where it does not conflict with local requirements. Where more stringent local,

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municipal, county, or state government requirements apply, such requirements shall be specified, and this standard shall be interpreted to supplement them.

Sec. 1.2 Drawings to Be Provided

Construction drawings for the foundation and tank shall be provided. Where foundation and tank designs are performed by separate parties, each party shall provide construction drawings in accordance with the particular portion of the tank facility for which they have design responsibility. For tanks with anchor bolts or base-setting rings, the tank designer shall provide details of the required embedment, water stops, additional concrete reinforcement, and minimum required concrete strength. Details of all bolted joints shall be referenced on the tank construction drawings.

SECTION 2: REFERENCES

This standard references the following documents. In their latest editions, or as specified, they form a part of this standard to the extent specified within the standard. In any case of conflict, the requirements of this standard shall prevail.

AA* ADM—*Aluminum Design Manual*, 2015.

AA ASD—*Aluminum Standards and Data*, 2013.

AA SPC-18—Standards for Aluminum Sand and Permanent Mold Castings, 15th Edition, 2008.

ACI† 301-10—Specifications for Structural Concrete.

ACI 318-14—Building Code Requirements for Structural Concrete and Commentary.

ACI 349-06—Code Requirements for Nuclear Safety-Related Concrete Structures.

AISI‡ 1010—Carbon Steel—Plates; Structural Shapes; Rolled Floor Plates; Steel Sheet Piling.

AISI S100—North American Specification for the Design of Cold-Formed Steel Structural Members.

AISI T-192—Steel Plate Engineering Data—Volume 1, Steel Tanks for Liquid Storage

* The Aluminum Association, 1400 Crystal Drive, Suite 430, Arlington, VA 22202.
† American Concrete Institute, 38800 Country Club Drive, Farmington Hills, MI 48331.
‡ American Iron and Steel Institute, 1140 Connecticut Ave., NW, Suite 705, Washington, DC 20036.

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AISI T-192—Steel Plate Engineering Data—Volume 2, Useful Information on the Design of Plate Structures

ANSI*/AISC† 360—Specification for Structural Steel Buildings.

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

ANSI/AWWA D104—Automatically Controlled, Impressed-Current Cathodic Protection for the Interior Submerged Surfaces of Steel Water Storage Tanks.

ANSI/AWWA D106—Sacrificial Anode Cathodic Protection Systems for the Interior Submerged Surfaces of Steel Water Storage Tanks.

ANSI/AWWA D108—Aluminum Dome Roofs for Water Storage Facilities.

API‡ SPEC 5L, 45th Edition, 2012—Specification for Line Pipe.

API SPEC 12B, 16th Edition, 2014—Specification for Bolted Tanks for Storage of Production Liquids.

ASCE§ 7-10—Minimum Design Loads for Buildings and Other Structures.

ASME¶ B16.5-13—Pipe Flanges and Flanged Fittings: NPS ½ through 24 Metric/Inch Standard.

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

ASTM A27/A27M—Standard Specification for Steel Castings, Carbon, for General Application.

ASTM A36/A36M—Standard Specification for Carbon Structural Steel.

ASTM A48/A48M—Standard Specification for Gray Iron Castings.

ASTM A53/A53M—Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless.

ASTM A105/A105M—Standard Specification for Carbon Steel Forgings for Piping Applications.

ASTM A106/A106M—Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service.

ASTM A123/A123M-15—Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products.

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

† American Institute of Steel Construction, 130 East Randolph, Suite 2000, 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, Two Park Avenue, New York, NY 10016.

** ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428.

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ASTM A139/A139M—Standard Specification for Electric-Fusion (Arc)-Welded Steel Pipe (NPS 4 and Over).

ASTM A153/A153M—Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware.

ASTM A181/A181M—Standard Specification for Carbon Steel Forgings, for General-Purpose Piping.

ASTM A194/A194M—Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both.

ASTM A216/A216M—Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High-Temperature Service.

ASTM A240/A240M—Standard Specification for Chromium and Chromium–Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications.

ASTM A283/A283M—Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates.

ASTM A307—Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60,000 PSI Tensile Strength.

ASTM A500/A500M—Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes.

ASTM A501/A501M—Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing.

ASTM A563-15—Standard Specification for Carbon and Alloy Steel Nuts.

ASTM A568/A568M—Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for.

ASTM A572/A572M—Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.

ASTM A656/A656M—Standard Specification for Hot-Rolled Structural Steel, High-Strength Low-Alloy Plate with Improved Formability.

ASTM A668/A668M—Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use.

ASTM A992/A992M—Standard Specification for Structural Steel Shapes.

ASTM A1011/A1011M—Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength.

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ASTM B695—Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel.

ASTM D395—Standard Test Methods for Rubber Property—Compression Set.

ASTM D412—Standard Test Methods for Vulcanized Rubber and Thermoplastic Elastomers—Tension.

ASTM D471—Standard Test Method for Rubber Property—Effect of Liquids.

ASTM D573—Standard Test Method for Rubber—Deterioration in an Air Oven.

ASTM D1171—Standard Test Method for Rubber Deterioration—Surface Ozone Cracking Outdoors or Chamber (Triangular Specimens).

ASTM D1229—Standard Test Method for Rubber Property—Compression Set at Low Temperatures.

ASTM D1751—Standard Specification for Preformed Expansion Joint Filler for Concrete Paving and Structural Construction (Nonextruding and Resilient Bituminous Types).

ASTM D2240—Standard Test Method for Rubber Property—Durometer Hardness.

ASTM D4752—Standard Test Method for Measuring MEK Resistance of Ethyl Silicate (Inorganic) Zinc-Rich Primers by Solvent Rub.

ASTM D5162—Standard Practice for Discontinuity (Holiday) Testing of Nonconductive Protective Coating on Metallic Substrates.

ASTM D5402—Standard Practice for Assessing the Solvent Resistance of Organic Coatings Using Solvent Rubs.

ASTM F1554—Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength.

ASTM F3125—Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions.

AWS* A5.1/A5.1M—Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding.

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

AWS D1.2/D1.2M—*Structural Welding Code—Aluminum.*

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

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AWS D1.6/D1.6M—*Structural Welding Code—Stainless Steel.*

ICC[*]—2015 International Building Code.

NACE[†] SP0188—Discontinuity (Holiday) Testing of New Protective Coatings on Conductive Substrates.

NFPA[‡] 22 2013—Standard for Water Tanks for Private Fire Protection.

NSF[§]/ANSI 61—Drinking Water System Components—Health Effects.

OSHA[¶]—Occupational Safety and Health Standards, 29 CFR, Part 1910.

SAE[**] J429—Mechanical and Material Requirements for Externally Threaded Fasteners.

SAE J995—Mechanical and Material Requirements for Steel Nuts.

SSPC[††]-SP 8—Pickling.

SSPC-SP 10/NACE No. 2—Near-White Metal Blast Cleaning.

SECTION 3: DEFINITIONS

The following definitions shall apply in this standard:

1. *BCL:* See *Water levels*.

2. *Capacity:* The net volume, in gallons (liters), that may be removed from a tank filled to the maximum operating level (MOL) and emptied to the bottom capacity level (BCL), unless otherwise specified. The BCL and MOL are defined below under *Water levels*.

3. *Constructor:* The party that furnishes the work and materials for placement and installation.

4. *Head range:* The vertical distance between the MOL and BCL.

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

6. *MOL*: See *Water levels.*

7. *MWL*: See *Water levels.*

[*] International Code Council, 500 New Jersey Avenue, NW, 6th Floor, Washington, DC 20001.
[†] NACE International, 1440 South Creek Drive, Houston, TX 77084.
[‡] National Fire Protection Association, 1 Batterymarch Park, Quincy, MA 02169.
[§] NSF International, 789 North Dixboro Road, Ann Arbor, MI 48105.
[¶] Occupational Safety and Health Administration, 200 Constitution Ave. NW, Washington, DC 20210.
[**] SAE International, 400 Commonwealth Drive, Warrendale, PA 15096.
[††] Society for Protective Coatings, 800 Trumbull Dr., Pittsburgh, PA 15205.

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

9. *Reservoir:* A ground-supported, flat-bottom cylindrical tank having a shell height equal to or smaller than its diameter.

10. *Standpipe:* A ground-supported, flat-bottom cylindrical tank having a shell height greater than its diameter.

11. *Tank:* A standpipe or a reservoir used for water storage.

12. *Water levels*: The following water levels are used in this standard:

12.1 Bottom capacity level (BCL): The water level above which the specified capacity is provided. The BCL shall be the water level in the tank shell when the tank is emptied through the specified discharge fittings, unless otherwise specified.

12.2 Maximum operating level (MOL): The specified maximum water level under normal operating conditions and the upper limit for calculating the specified operating capacity (head range) of the tank. The MOL shall be taken as the maximum water level (MWL), unless otherwise specified.

12.3 Maximum water level (MWL): The water level defined by the flow line of the overflow pipe or lip of the weir unless otherwise specified.

SECTION 4: MATERIALS

Sec. 4.1 General

All materials to be incorporated in any structure to meet the provisions of this standard shall be new, previously unused, and in first-class condition and shall comply with all of the requirements of this standard. Steel materials of unidentified analysis may be used if they are tested and found to comply with all of the physical, dimensional, and chemical requirements of a material that is acceptable for use under this standard. A report showing the test results shall be provided when mill test reports are specified.

Materials shall comply with the Safe Drinking Water Act and federal regulations for potable water and wastewater systems as applicable.

Sec. 4.2 Bolts, Nuts, and Anchor Bolts

4.2.1 *Carbon steel bolts and nuts.* Carbon steel bolts shall conform to the requirements of ASTM A307, ASTM F3125, SAE J429 grade 2, SAE J429

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grade 5, SAE J429 grade 8, or API SPEC 12B. Carbon steel nuts shall be suitable for use with selected bolt and shall conform to API SPEC 12B, SAE J995 grade 2, SAE J995 grade 5, SAE J995 grade 8, ASTM A194/A194M, or ASTM A563.

4.2.2 *Galvanizing.* Hot-dip galvanizing of fasteners shall conform to ASTM A153/A153M, class C. Mechanical galvanizing of fasteners shall conform to ASTM B695, class 50. Mechanical galvanizing is permitted for ASTM F3125, SAE J429 grade 8, and SAE J995 grade 8 fasteners, provided that proper procedures are followed to prevent or eliminate hydrogen embrittlement. ASTM F3125, SAE J429 grade 8, and SAE J995 grade 8 high-strength fasteners shall not be hot-dip galvanized.

4.2.3 *Carbon steel anchor bolts.* Carbon steel anchor bolts shall conform to the requirements of ASTM A36/A36M, or ASTM F1554 grade 36 or 55.

Sec. 4.3 Foundation-Reinforcing Steel

Reinforcing steel in foundations shall comply with the requirements of ACI 318.

Sec. 4.4 Plates and Sheets

Carbon steel plate and sheet materials shall be open-hearth, electric-furnace, or basic-oxygen-process steel conforming to any of the following ASTM specifications: A36/A36M; A283/A283M, grade C or D; A1011/A1011M, grade 30, 33, 36, 40, 45, 50, 55, 60, 65, or 70; A572/A572M, grade 42, 50, 55, 60, or 65; A656/A656M, grade 50, 60, or 70.

Plates and sheets may be provided on the weight basis, with permissible underrun according to the tolerance table for plates ordered to weight published in ASTM A6/A6M. For sheets ordered to weight published in ASTM A568/A568M, no underrun is permitted.

Materials conforming to ASTM A1011/A1011M with a designation CS, DS, or UHSS shall not be used. No grade, type, or class of ASTM A1011/A1011M material shall be used where the ratio of published minimum yield strength to published minimum tensile strength exceeds 0.85. For ASTM A656/A656M grade 60 and grade 70 material, the ratio of the actual yield strength to the actual tensile strength shall not exceed 0.85 as confirmed by mill test reports.

Steel grades designating a published minimum yield strength of 50,000 psi or greater shall not be used for shell sheets or plates on 15-ft or smaller-diameter tanks that have form-flanged connections.

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Sec. 4.5 Structural Shapes

4.5.1 *Carbon steel.* Hot-rolled carbon steel structural shapes for use under the provisions of this standard shall conform to ASTM A6/A6M. Material shall conform to ASTM A36/A36M, A572/A572M grade 50, A992/A992M, or AISI 1010. Additionally, ASTM A992/A992M is acceptable for carbon steel wide flange and other structural shapes.

4.5.2 *Aluminum.* Aluminum shapes of a suitable alloy for load and service requirements may be used for portions of the tank not in contact with water. The design of all aluminum members shall be in accordance with AA ADM (Allowable Stress Design) and to the loads specified in Section 5 of this standard. Aluminum in contact with dissimilar metals shall be properly isolated to prevent galvanic corrosion.

4.5.3 *Tubular.* Tubular steel structural shapes may be used for columns and other structural components. Carbon steel tubular structural members shall comply with one of the following specifications:

1. Cold-formed square and rectangular carbon steel structural tubing shall comply with ASTM A500/A500M.

2. Hot-formed square and rectangular carbon steel tubing shall comply with ASTM A501/A501M.

4.5.3.1 Structural tubing with a circular cross section may be manufactured from plates of any of the specifications permitted in Sec. 4.4, provided the welding and other manufacturing processes are in compliance with all sections of this standard.

4.5.3.2 Carbon steel pipe may be used as tubular structural members, provided it complies with ASTM A139/A139M, grade B; ASTM A53/A53M, type E or S, grade B; ASTM A106/A106M, grade B; or API SPEC 5L, grade B, and provided the minimum thickness of any such material shall comply with the design requirements regardless of the thickness tolerances in any of these specifications. Some pipe specifications allow thickness underruns as high as 12.5 percent. The appropriate specification shall be consulted for allowable underrun and for adjustments made in thickness to ensure that minimum design thicknesses are met.

Sec. 4.6 Castings

Iron castings shall conform to ASTM A48/A48M, class 30. Steel castings shall conform to ASTM A216/A216M, grade WCB, or ASTM A27/A27M, grade 60-30 (full annealed). Aluminum castings shall conform to AA SPC-18.

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Sec. 4.7 Forgings

4.7.1 *Forgings from plate and sheet materials.* Forgings from plate and sheet materials shall conform to the plate and sheet materials permitted under Sec. 4.4.

4.7.2 *Forgings from other than plate or sheet materials.* Forgings from other than plate or sheet materials shall conform to ASTM A668/A668M, class E.

4.7.3 *Forged and rolled pipe flanges.* Carbon steel forged and rolled pipe flanges shall conform to ASTM A181/A181M, class 60; ASTM A105/A105M; or ASME B16.5.

Sec. 4.8 Electrodes

Manual, shielded-metal, arc-welding electrodes shall conform to the requirements of AWS A5.1/A5.1M. Welding electrodes shall be of any E60XX or E70XX classification suitable for the electric current characteristics, the position of welding, and other conditions of intended use. Welding electrodes for other welding processes shall conform to applicable AWS specifications for filler metal.

Sec. 4.9 Pipe and Fittings for Fluid Conductors

Inlet, outlet, overflow, and other pipes, and all fittings for fluid use shall be specified. Carbon steel pipe shall conform to ASTM A53/A53M type E or S, grade B; ASTM A106/A106M; or API SPEC 5L. When specified, pipe for fluid conductors smaller than 6-in. nominal diameter may be constructed of stainless steel and shall conform to ASTM A240/A240M 300 series. Other types of pipe may be specified, provided they conform to recognized national or industry standards. Unless otherwise specified, joints may be either screwed or flanged.

Sec. 4.10 Gaskets and Sealants

The manufacturer shall use gaskets or sealants or a combination of both in accordance with the following requirements. Gasket and sealant material in contact with stored potable water shall be tested and certified for potable water contact in accordance with the requirements of NSF/ANSI 61, when specified.

4.10.1 *Gaskets.* Gasket material shall be of adequate tensile strength and resilience to obtain a leakproof seal at all seams and joints. Gasket material shall be resistant to weather and ozone exposure as designated by ASTM D1171. Physical requirements are described in Table 1.

4.10.2 *Sealants.* Sealants shall comply with the following:

1. Resistance to temperature. The sealant shall remain flexible when in continuous operation over a temperature range of –40°F to +170°F (–40°C to +77°C).

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Table 1 Physical requirements for gasket material*

Description	Strip and Extruded Gasket Material
Tensile strength, initial, psi minimum, ASTM D412	1,200 psi (8.3 MPa)
Tensile strength after oven aging as percent of initial, minimum, ASTM D573	70%
Tensile strength after immersion in distilled water as percent of initial, minimum, ASTM D471	60%
Ultimate elongation, initial, percent of minimum, ASTM D412	175%
Ultimate elongation after oven aging as percent of initial, minimum, ASTM D573	70%
Hardness, Shore A, ASTM D2240	75±5
Hardness change Shore A, after oven aging, ASTM D573	7
Compression set as maximum percent of original, after oven aging, ASTM D395	40%
Low-temperature compression set as maximum percent of original, ASTM D1229	60%
Tear strength, pounds per inch	160 lb/in. (28 kN/m)

* Dimensions and tolerances shall be as specified by the manufacturer for the specific tank requirements.

2. Weatherability. The sealant shall be resistant to hardening and cracking. The sealant shall be essentially solid and contain no plasticizers or extenders that could cause shrinkage due to weathering. The sealant shall be resistant to ozone and ultraviolet light.

3. Chemical resistance. The sealant shall be suitable for contact with potable water without chemical extraction to water and shall not swell or degrade under normal water-storage conditions.

4. Material specification. The sealant shall be suitable for use on potable water-contact surfaces and shall meet NSF/ANSI 61 requirements where in contact with stored potable water, when specified.

5. Primers for sealant. Some sealant materials require the use of a primer on metal or glass for maximum adhesion. Most of these primers contain a volatile solvent. After evaporation of the solvent, the primer shall meet NSF/ANSI 61 requirements where in contact with stored potable water, when specified.

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SECTION 5: GENERAL DESIGN

Sec. 5.1 Types of Joints

5.1.1 *Bolted joints.* All vertical, horizontal, shell-to-roof, and shell-to-bottom plates or sheets shall be field-bolted. Bolt holes shall be shop-punched or drilled for field assembly. The bolted joints between roof, shell, and bottom sheets and plates that are required to contain water or to be weathertight shall be sealed with suitable gasket material, sealant, or gasket material and sealant as required to make a watertight joint (see Sec. 4.10). The tank manufacturer's erection procedures shall be followed.

5.1.2 *Welded joints.* Welding may be used to join shop-fabricated subassemblies that are subsequently bolted into place in the field. Vertical joints in the tank shell shall not be welded.

Sec. 5.2 Design Loads

The following loads shall be considered in the design of tank structures and foundations.

5.2.1 *Risk category.* Risk category is a categorization of the structure for the determination of snow, wind, and seismic loads based on the risk associated with unacceptable performance. The structure shall be classified one of the following risk categories and the risk category shall be specified. Risk Category IV shall be used if none is specified.

5.2.1.1 Risk Category II. All structures other than those included in Risk Categories III and IV.

5.2.1.2 Risk Category III. Structures where failure could pose a substantial risk to human life. Structures not included in Risk Category IV that have the potential to cause substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure.

5.2.1.3 Risk Category IV. Structures designated as essential facilities. Structures required to maintain the functionality of other Risk Category IV structures.

5.2.2 *Dead load.* Dead load shall be the estimated weight of all permanent construction and fittings. The unit weights used shall be 490 lb/ft^3 (7,850 kg/m^3) for steel and 144 lb/ft^3 (2,310 kg/m^3) for concrete.

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5.2.3 *Water load.* Water load shall be the weight of all of the liquid when the tank is filled to maximum water level (MWL). Unit weight used for water shall be 62.4 lb/ft^3 (1,000 kg/m^3), except as noted in Section 14 for seismic design.

5.2.4 *Snow load.* Snow load shall be in accordance with ASCE 7, except as modified herein.

5.2.4.1 Ground snow load. Ground snow load p_g shall be in accordance with ASCE 7, Figure 7-1 and Table 7-1. Ground snow load shall be specified for areas designated C_S and for sites at elevations above the limits indicated in ASCE 7, Figure 7-1. Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval). Snow loads are zero for Hawaii, except in mountainous regions. Snow load shall be specified for mountainous regions of Hawaii.

5.2.4.2 Flat roof snow load. Flat roof snow load p_f shall be determined using the equation

$$p_f = 0.7 C_e C_t I_s p_g \qquad \text{(Eq 5-1)}^*$$

where:

p_f = flat roof snow load, in pounds per square foot
C_e = exposure factor
= 0.9 for wind Exposure C
= 0.8 for wind Exposure D
C_t = thermal factor
= 1.2 for unheated and open structures
I_s = importance factor for snow load
= 1.0 for Risk Category II structures
= 1.1 for Risk Category III structures
= 1.2 for Risk Category IV structures
p_g = ground snow load, in pounds per square foot

* For equivalent metric equation, see Sec. 5.11.

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5.2.4.3 Sloped roof snow load. Sloped roof snow load p_s shall be zero for portions of the roof with a roof angle θ_r equal to or greater than 70 degrees. For portions of the roof with a roof angle less than 70 degrees, the sloped roof snow load shall be determined using the following equations

$$C_s = \frac{70 - \theta_r}{55}, 0 \leq C_s \leq 1 \quad \text{(Eq 5-2)}$$

$$p_s = C_s p_f \quad \text{(Eq 5-3)}$$

where:

p_s = sloped roof snow load, in pounds per square foot

C_s = slope factor based on cold roofs with unobstructed slippery surfaces that will allow snow to slide

θ_r = roof angle measured as the angle between the tangent of the roof and the horizontal at the point of consideration, in degrees

5.2.4.4 Minimum snow load for sloped roofs. The minimum snow load shall only apply to monoslope and conical roofs with a roof angle θ_r less than 15 degrees and portions of double-curved roofs with a roof angle less than 15 degrees. Where the ground snow load p_g is equal to or less than 20 lb/ft^2 (0.96 kN/m^2), the minimum snow load shall be $I_s p_g$. Where the ground snow load is greater than 20 lb/ft^2 (0.96 kN/m^2), the minimum snow load shall be $20 I_s$.

5.2.5 *Roof live load.* The minimum roof live load shall be 15 lb/ft^2 (720 N/m^2).

5.2.6 *Wind load.* Refer to Section 15 of this standard for wind load design. When so specified, wind loads may be omitted for structures completely enclosed within buildings.

5.2.7 *Seismic load.* Structures shall be designed for seismic loads as defined in Section 14 of this standard. Exception: Structures located where mapped spectral response acceleration at 1-s period S_1 is less than or equal to 0.04g and the mapped short period spectral response acceleration S_s is less than or equal to 0.15g do not require design for seismic loads. See Section 14 for definitions.

5.2.8 *Platform and ladder load.* A vertical load (and only one such load in each case) shall be applied as follows: 1,000 lb (4.4 kN) to each platform; 500 lb (2.2 kN) to any 10 ft^2 (0.93 m^2) area on the tank roof; 500 lb (2.2 kN) on each vertical section of the ladder. All structural parts and connections shall be

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properly proportioned to withstand such loads. The aforementioned load need not be combined with the snow load specified in Sec. 5.2.4, but it shall be combined with the dead load. The platform and roof plating may deflect between structural supports in order to support the loading.

Sec. 5.3 Design Criteria

With the exception of other criteria specifically provided for elsewhere in this standard, the structural design of all standpipes and reservoirs shall be in compliance with the following: AISI S100 (ASD provisions) and ANSI/AISC 360 (ASD provisions).

5.3.1 *Allowable stresses increased by one-third.* Except as specified otherwise, for members subjected to loads produced by wind or seismic forces, acting alone or in combination with the design dead and water loads, the allowable stresses may be increased by one-third, provided the required section computed on this basis is not less than that required for the design dead and water load computed without the one-third allowable stress increase.

5.3.2 *Effect of glass-fused coatings firing process on steel strength*. The design of structural components with glass-fused-to-steel coatings shall take into account the reduction in strength properties of the steel due to the firing process of the glass coatings, and details of such effects shall be submitted when specified. The effect of the glass-fused-to-steel coating process shall be assessed and monitored over a period of time using a regular and documented testing program from which steel strength properties can be predicted with a 95 percent confidence level. The 95 percent confidence level refers to the 95 percent confidence interval, which is the statistically calculated range within which there is a 95 percent probability that the true value of a parameter will fall.

5.3.2.1 Reduced strength properties for components with glass-fused-to-steel coatings. The modified steel properties F'_y and F'_u shall be the minimum values established for the particular specification and grade of steel through the regular and documented testing program as identified in Sec. 5.3.2 or, alternatively, where regular and documented testing is not carried out, the values of F'_y and F'_u shall be taken as 70 percent of the minimum published yield strength, F_y, and 70 percent of the minimum published tensile strength, F_u, respectively, of the selected material.

5.3.2.2 Application of reduced strength properties. For components with glass-fused-to-steel coatings, design of the components shall utilize the lesser of the applicable modified steel properties F'_y and F'_u and the minimum published

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yield strength and minimum published tensile strength, F_y and F_u, respectively. Reduced strength properties only apply to structural elements of the tank that have undergone the glass-fused-to-steel coating process.

Sec. 5.4 Tank Shell

In the design of the tank shell, the hydrostatic water pressure at the lower edge of each ring of sheets or plates in the tank shell shall be assumed to act undiminished on the entire area of the ring.

5.4.1 *Shell thickness.* When the net tensile stress governs, the thickness of cylindrical shell plates stressed by pressure of the tank contents shall be calculated by the formula

$$t = \frac{2.6\,HDSG}{f_t(S - d)} \qquad \text{(Eq 5-4)}^*$$

where:

t = shell plate thickness in inches
H = height of liquid from the top capacity line just to overflow to the bottom of the shell course being designed in feet
D = tank diameter in feet
S = bolt spacing in line perpendicular to line of stress in inches
G = specific gravity of liquid (1.0 for water)
f_t = allowable tensile stress in pounds per square inch (Sec. 5.5.3)
d = bolt-hole diameter in inches

5.4.2 *Compressive stress.* The allowable compressive stress in each ring of sheets or plates under wind or earthquake loading combined with dead load shall be determined by the formula

$$f_s = 15{,}000\left(\frac{2}{3}\right)\left(100\frac{t}{R}\right) \times \left[2 - \left(\frac{2}{3}\right)\left(100\frac{t}{R}\right)\right] \leq 15{,}000 \qquad \text{(Eq 5-5)}^*$$

where:

f_s = allowable compressive stress in pounds per square inch
t = shell thickness in inches
R = shell radius in inches

* For equivalent metric equation, see Sec. 5.11.

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Sec. 5.5 Bolted Joints

In the design of bolted joints, the effect of the gasket and sealant shall be neglected, provided the compressed thickness of the gasket or sealant does not exceed 1⁄16 in. (1.6 mm).

5.5.1 *Minimum spacing.* The center-to-center distance between bolts shall not be less than 2*d*, where *d* is the diameter of the bolt, in inches (millimeters). The distance from the center of any bolt to an edge or seam shall not be less than 0.75 in. In no case shall the center-to-center or edge-to-center distance be less than

$$\frac{P}{0.6 F_y t} \text{ inches (m)} \qquad \text{(Eq 5-6)}$$

where:

P = force transmitted by the bolt, lb (kN)
F_y = published minimum yield strength of the sheet or plate, psi (MPa)
t = thickness of the thinner sheet or plate in the joint connection, in. (mm)

5.5.2 *Multiple bolt lines.* When multiple bolt lines are used, the effective net section area shall not be taken as greater than 85 percent of the gross area.

5.5.3 *Tension on the net section.* The tensile stress on the net section of a bolted connection shall not exceed the lesser of the values determined by the following formulas:

$$f_t = 0.6F_y(1.0 - 0.9r + 3rd/s) \leq 0.6F_y \qquad \text{(Eq 5-7)}$$

or

$$f_t = 0.40F_u \qquad \text{(Eq 5-8)}$$

where:

f_t = allowable tensile stress, psi (MPa)
F_y = published minimum yield strength of the sheet or plate, psi (MPa)
r = force transmitted by the bolt or bolts at the section considered, divided by the tensile force in the member at that section. If r is less than 0.2, it may be taken to equal zero.
d = diameter of bolt in inches (mm)
s = spacing of bolts perpendicular to line of stress in inches (mm)
F_u = published minimum ultimate strength of the sheet or plate, psi (MPa)

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5.5.4 *Hole bearing stress.* The hole bearing stress on the area $d \times t$ shall not exceed $1.35F_y$. The symbols d and F_y are as defined in Sec. 5.5.3; t is the thickness of the sheet or plate under consideration.

5.5.5 *Bolt shear.* Shear on bolts in live and dead loads shall not exceed the value as determined from the formula

$$f_v = F_u(0.6)(0.9) / 2.2 \leq 0.25F_u \quad \text{(Eq 5-9)}$$

where:

f_v = allowable shear stress to the affected area, whether tensile stress area or gross area, psi (MPa)

F_u = published minimum tensile strength of bolt, psi (MPa)

5.5.6 *Bolt tension.* The tensile stress on the tensile stress area of bolts, other than anchor bolts, shall not exceed the lesser of the following:

$$f_t = 0.6F_y \quad \text{(Eq 5-10)}$$

or

$$f_t = F_u / 2.2 \quad \text{(Eq 5-11)}$$

where:

f_t = allowable tensile stress of the bolt, psi (MPa)

F_y = published minimum yield strength of the bolt, psi (MPa)

F_u = published minimum tensile strength of the bolt, psi (MPa)

5.5.7 *Tensile Straps*. Horizontal joints on tanks with formed flange construction could require tensile straps to transfer vertical tension loads. The design procedure in Sec. 15.5 shall be utilized to determine if tensile straps are required for these joints to resist forces generated under load combinations including either wind or seismic loads, or both.

Sec. 5.6 Weld Design

5.6.1 *Structural joints*. Welded structural joints shall be proportioned so that the stresses on a section through the throat of the weld, exclusive of weld reinforcement, do not exceed the following percentages of the allowable tensile stress of the structural material joined.

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5.6.1.1 Groove welds. Tension, 85 percent; compression, 100 percent; shear, 75 percent.

5.6.1.2 Fillet welds. Transverse shear, 65 percent; longitudinal shear, 50 percent; varying shear for circular weldments, 60 percent.

NOTE: Stress in a fillet weld shall be considered as shear on the throat, for any direction of the applied load. The throat of a fillet weld shall be assumed as 0.707 times the length of the shorter leg of the fillet weld having a flat or slightly convex profile.

Sec. 5.7 Roof Supports

Roof supports or stiffeners, if used, shall be designed in accordance with the current specifications of ANSI/AISC 360-10, with the following stipulations or exceptions:

1. The roof sheet shall be assumed to provide the necessary lateral support of roof rafters from the friction between the roof plates and rafter compression flange, with the following exceptions:
   a. Trusses and open-web joists used as rafters.
   b. Rafters having a nominal depth greater than 15 in. (380 mm).
   c. Rafters having a slope greater than 2 in 12.
2. The roof rafter and purlin depth may be less than $f_b/600{,}000$ ($f_b/4{,}140$) times the span length, provided the roof slope is ¾ in 12 or greater. The symbol f_b is the bending unit stress in pounds per square inch (MPa), equal to the bending moment divided by the section modulus of the member.
3. The maximum slenderness ratio KL/r for the column supporting rafters shall be 175. The notation L is the laterally unsupported length of the column, in inches (mm), and r is the least radius of gyration, in inches (mm).
4. Roof trusses shall be placed above the top capacity level in climates where ice may form. No part shall project below the top capacity level.
5. Roof rafters shall be placed above the top capacity level. No part shall project below the top capacity level.
6. The maximum spacing between roof supports shall be determined by the formula

$$L = (300F_y t^2/W)^{1/2} \leq 60 \qquad \text{(Eq 5-12)}^*$$

where:

L = maximum centerline spacing, in.

F_y = published minimum yield strength of the roof sheet, psi

* For equivalent metric equation, see Sec. 5.11.

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t = roof sheet thickness, in.

W = roof dead load plus live load acting on roof surface, lb/ft^2

7. Self-supporting dome roof constructed of aluminum shall comply with the requirements of ANSI/AWWA D108.

Sec. 5.8 Steel Thickness

Steel thicknesses shall comply with the following:

1. Roof sheets having a slope of 1 in 2.75 or greater shall have a minimum thickness of 0.070 in. (1.8 mm) where the tank diameter does not exceed 35 ft (10.7 m), and a minimum thickness of 0.094 in. (2.4 mm) where the tank diameter is greater than 35 ft (10.7 m).

2. Roof sheets having a slope less than 1 in 2.75, regardless of tank diameter, shall have a minimum thickness of 0.094 in. (2.4 mm).

3. The minimum thickness of the bottom sheets shall be 0.094 in. (2.4 mm).

4. The maximum thickness of formed flange shell plates shall be 0.3125 in. (7.9 mm); the minimum thickness shall be 0.094 in. (2.4 mm).

5. The maximum thickness of flat panel shell plates shall be 0.50 in. (12.7 mm); the minimum thickness shall be 0.094 in. (2.4 mm).

6. Tank components fabricated from ASTM A1011 material shall be less than 0.230 in. thickness.

7. Plates and sheets shall not be stacked or laminated together to form multiple layers to achieve a given design thickness.

Sec. 5.9 Foundation Anchor Bolts

5.9.1 *General.*

5.9.1.1 Required anchorage. Anchor bolts shall be provided when the wind or seismic loads exceed the limits for self-anchored tanks. (Refer to Section 14 for seismic loads and Section 15 for wind loads.)

5.9.1.2 Spacing.

5.9.1.2.1 Maximum anchor bolt spacing shall be 10 ft (3 m). Anchored tanks shall have a minimum quantity of five anchor bolts.

5.9.1.2.2 The pullout value of the anchor bolts shall be adjusted for anchor spacing and edge distance.

5.9.1.2.3 Anchor bolts shall be evenly spaced except where interference with tank openings or tank accessories does not permit. At locations where tank openings or tank accessories interfere with one or two anchors, no more than two anchors adjacent to the interference may be moved a maximum of 50 percent of

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the uniform spacing. A special analysis is required at locations where tank openings or tank accessories interfere with more than two anchors.

5.9.1.2.4 Allowable stresses for anchor bolts, anchor chairs, and anchor attachments may be increased for wind and seismic loads as permitted in Sec. 5.3.1.

5.9.2 *Strength of anchor bolt attachment.* Anchor bolt attachments shall be designed such that the bolt yields before the anchor chair or anchor attachment fails. This check shall be made in both the as-built condition and, if a corrosion allowance is specified, the corroded condition of the anchor, anchor chair, attachment, and tank shell.

5.9.3 *Unit stresses.*

5.9.3.1 Anchor bolt material is listed in Sec. 4.2.

5.9.3.2 The allowable unit tension stress of the anchor bolt, based on the minimum root area of the bolt, in accordance with ANSI/AISC 360-10, shall be the lesser of 0.6 times the published minimum yield stress or 0.33 times the published minimum ultimate stress of the material. The minimum published yield strength shall not exceed 70 percent of the minimum published tensile strength for anchor bolts made from material other than those listed in Sec. 4.2.

5.9.4 *Anchor bolt requirements.*

1. The minimum anchor bolt diameter shall be ¾ in. (19 mm).

2. All carbon steel anchor bolts, nuts, and washers shall be hot-dip zinc coated in accordance with ASTM F1554.

3. Anchor bolt embedment shall terminate in a head, nut, or washer plate except as allowed in Sec. 5.9.4 (8).

4. At the top of anchor bolts, lock nuts shall be provided, or the threads shall be peened to prevent loosening of the nuts.

5. To provide for variations in the foundation elevations, the anchor bolt projection above the top of the foundation concrete shall be specified to provide a 2-in. (51-mm) nominal projection of the threaded ends of the anchor bolts above the design elevation of the tops of the anchor bolt nuts (or lock nuts).

6. The minimum actual projection shall be sufficient to allow peening of the threads, if peening is used. Otherwise, the threaded end of the anchor bolt shall not be lower than flush with the top of the anchor bolt lock nut.

7. Refer to Sec. 13.6.3 for tolerances.

8. Cast-in-place anchor bolts are recommended. However, field-installed anchor bolts that are not cast-in-place, such as anchor bolts utilizing epoxy and wedge-type anchor systems, may be used, provided the anchor system manufacturer's

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recommendations and limitations for loading, spacing, edge distance, installation procedures, environmental conditions, any seismic and wind provisions, creep considerations, and other requirements are strictly adhered to. Foundation must be of sound condition and properly designed to accept the anchor system. To verify the anchor system capacities, pullout and other strength tests shall be performed, if specified.

5.9.5 *Design of anchor chairs.*

5.9.5.1 Anchor bolt chairs. Anchor bolt chairs shall be designed in accordance with AISI T-192 Steel Plate Engineering Data—Volume 1, Useful Information on the Design of Plate Structures. Alternate anchor-chair configurations may be used, provided they are proven by test or calculation. Allowable stresses shall be calculated in accordance with Sec. 5.4.2, Sec. 5.5.3, Sec. 5.5.5, and Sec. 5.6 as applicable.

5.9.6 *Design for resistance to base shear.*

5.9.6.1 Design loads. The net base shear V_{NET} to be resisted by the anchorage shall be that portion of the calculated base shear that exceeds the calculated frictional resistance. See Sec. 14.3.4.5. Only the effective portion of the shear anchorage system shall be considered to resist V_{NET} for any direction of ground motion.

5.9.6.2 Special considerations. When tanks are anchored for uplift loads using anchor bolts with anchor chairs, special details or separate shear resistance must be provided when V_{NET} exceeds zero.

5.9.7 *Embedment and reinforcement requirements.* Anchor embedment and reinforcement shall meet the following requirements:

1. Design of anchor embedment shall be in accordance with the International Building Code (IBC), ACI 318, or ACI 349.
2. Provide at least 3-in. (76-mm) clear cover between the anchor and bottom of the foundation.
3. Installation of anchor bolts greater than 2 in. (51 mm) in diameter shall comply with the special reinforcement requirements of ACI 318.
4. Anchor embedment shall be designed such that the anchor yields before the embedment fails or pulls out of the foundation. This check shall be made for the as-built condition of the anchor.
5. Bond stress of the anchor shall not be considered in determining the load capacity of the anchor embedment.

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5.9.8 *Design loads.*

5.9.8.1 Anchor bolts. Anchor bolts shall be designed for the maximum effect of the design uplift forces, including the wind uplift force and the seismic uplift force considering applicable load combinations and allowable stress levels. The design uplift forces represent the net uplift force to be resisted by the anchor after consideration for any reductions resulting from the dead weight of the structure. When the calculated net uplift force, P_W, results in a negative value for wind load, no uplift anchorage is required; see Sec. 14.3.4.1 for seismic anchorage requirements. When anchor bolts are required or provided, the design tension load per anchor bolt shall be calculated as follows:

$$P_W = \frac{4M_W}{ND_{ac}} - \frac{W'}{N} \qquad \text{(Eq 5-13)}^*$$

$$P_S = \frac{4M_S}{ND_{ac}} - \frac{(1 - 0.4A_v)W'}{N} \qquad \text{(Eq 5-14)}^*$$

where:

P_W, P_s = design tension force per anchor bolt for wind and seismic loads, respectively, lb

D_{ac} = diameter of anchor bolt circle, ft

N = number of anchor bolts

M_W, M_s = wind and seismic overturning moment, respectively, ft-lb

W' = total weight of tank shell, roof dead-load reaction on shell, and permanent accessories available to resist uplift, lb

A_v = vertical design accelerations from Sec. 14.3.4.3 stated as a multiple (decimal) of g

5.9.8.2 Anchor chairs and anchor attachments. Anchor chairs and anchor attachments to the tank shell shall be designed using the following loads:

1. Wind: P_W from Eq 5-13.
2. Seismic: Lesser of the anchor bolt yield capacity based on the area at the root of the threads and:

$$P_a = \frac{16M_S}{ND_{ac}} - \frac{(1 - 0.4A_v)W'}{N} \qquad \text{(Eq 5-15)}^*$$

* For equivalent metric equation, see Sec. 5.11.

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where:

P_a = design tension force per anchor bolt for anchor chair and anchor attachment for seismic loads, respectively, lb

Sec. 5.10 Reinforcement Around Openings

All welded or bolted connections greater than 2 in. (51 mm) in diameter in the tank shell and other locations that are subject to hydrostatic pressure and/or structural loads shall be reinforced. The reinforcement may be the flange of a fitting, an additional ring of metal, a thicker plate, or any combination of these.

5.10.1 *Tank shell.* The amount of reinforcement for an opening in the tank shell shall be computed as follows: The minimum cross-sectional area of the reinforcement shall not be less than the product of the maximum dimension of the hole cut in the tank shell and any bolt holes in the line perpendicular to the direction of the maximum stress and the required shell thickness. The cross-sectional area of the reinforcement shall be measured perpendicular to the direction of maximum stress coincident with the maximum dimension of the opening (100 percent reinforcement). Except for flush-type cleanout fittings, all effective reinforcement shall be made within a distance equal to the maximum dimension of the hole in the shell. The direction of reinforcement shall be perpendicular to the maximum stress. Shell plate thickness in excess of that actually required to retain and support the liquid contents for the specified loads, exclusive of that which is required by the shell bolted joint design, may be used as reinforcement area. The reinforcement shall be in either direction from the centerline of the shell opening.

5.10.2 *Fittings.* In the computation of the net reinforcing area of a fitting having a neck (such as a boilermaker's flange or a manhole saddle), the following portions of the neck may be considered as part of the area of reinforcement:

1. That portion extending outward from the outside surface of the shell for a distance equal to four times the neck wall thickness or, if the neck wall thickness is not uniform within that distance, to the point of transition.
2. That portion lying within the shell thickness.
3. If the neck extends inwardly, that portion extending inward from the inside surface of the shell for a distance as specified in item 1 in this list.

5.10.2.1 The aggregate strength of the weld or bolt group attaching a fitting to the shell or any intervening reinforcing plate, or both, shall at least equal the proportion of the forces passing through the entire reinforcement that is computed to pass through the fitting.

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5.10.2.2 The attachment weld joining the flanged fitting or reinforcing plate to the shell shall be considered effective along the outer periphery only for the parts lying outside of the area bounded by parallel lines drawn tangent to the shell opening perpendicular to the direction of maximum stress. The outer peripheral welding, however, shall be applied completely around the reinforcement. All of the inner peripheral weld shall be considered effective. The outer peripheral weld shall be of a size equal to the thickness of the shell or reinforcing plate, whichever is less.

5.10.2.3 Manhole necks, nozzle necks, reinforcing plates, and shell openings that have sheared or oxygen-cut surfaces shall have uniform and smooth surfaces, with the corners rounded, except where these surfaces are fully covered by attachment welds.

Sec. 5.11 Equivalent Metric Equations

Metric equivalents of equations presented in Section 5 are as follows:

Equation Number	Equivalent Metric Equation	Variable	Metric Units
5-4	$t = \frac{4.9HDSG}{f_t(S-d)}$	t H D S f_t d	mm m m mm MPa mm
5-5	$f_s = 103\left(\frac{2}{3}\right)\left(100\frac{t}{R}\right)\left(2-\left(\frac{2}{3}\right)\left(100\frac{t}{R}\right)\right) \leq 103$	f_s t R	MPa mm mm
5-12	$L = \left(2{,}000F_y\frac{t^2}{W}\right)^{1/2} \leq 1{,}520$	L F_y t W	mm MPa mm kPa
5-13	$P_w = \frac{4M_w}{ND_{ac}} - \frac{9.81W'}{N}$	P_W M_W D_{ac} W'	N N-m m kg
5-14	$P_s = \frac{4M_s}{ND_{ac}} - \frac{(1-0.4A_v)9.81W'}{N}$	P_s M_s D_{ac} W'	N N-m m kg
5-15	$P_a = \frac{16M_s}{ND_{ac}} - \frac{(1-0.4A_v)9.81W'}{N}$	P_s M_s D_{ac} W'	N N-m m kg

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SECTION 6: SIZING OF TANKS

Sec. 6.1 Standard Capacities, Diameters, and Heights

The standard capacities for tanks shall be as published by the manufacturer and shall be calculated to the nearest 1,000 gal (5 m^3). The required diameter, shell height, and capacity shall be specified, with an allowable variation to conform with the manufacturer's standard modular sizes and capacities.

SECTION 7: ACCESSORIES FOR TANKS

Sec. 7.1 Shell Manholes

One manhole, unless otherwise specified, shall be provided in the first ring of the tank shell. In tanks with one manhole, a sheet opposite the manhole may be removed for additional ventilation if required for inspections or recoating. If any manhole cover weighs more than 50 lb (23 kg), a hinge or davit shall be provided.

7.1.1 *Size and shape.* Manholes may be either circular, 24 in. (610 mm) in diameter; square, 24 in. (610 mm) × 24 in. (610 mm); or elliptical, 18 in. (460 mm) × 22 in. (560 mm), minimum size. Flush rectangular manholes with a minimum length of 24 in. (610 mm) in the shortest direction and a maximum length of 48 in. (1,220 mm) in the longest direction are also acceptable. Cutouts for rectangular manholes must have a minimum 6-in. (150-mm) radius at the corners.

7.1.2 *Reinforcing.* The shell plate where the manhole is located shall be reinforced to comply with Sec. 5.10, and all portions of the manhole, including the bolting, the cover, and reinforcement of the neck, shall be designed to withstand the weight and pressure of the tank contents.

Sec. 7.2 Pipe Connections

The quantity, size, type, and location of all connecting piping to the tank shall be specified as noted in the foreword, Sec. III.A.5.c.

7.2.1 *Silt stop.* If a removable silt stop is specified, it shall be at least 4 in. (100 mm) in height, and the fitting or piping connection shall be flush with the tank floor when the stop is removed. If a removable silt stop is not required, the fitting or connecting pipe, or both, shall extend above the floor by at least 4 in. (100 mm).

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7.2.2 *Shell connections.* Shell connections may be specified, provided adequate provisions are made to protect the pipe from freezing and provided adequate pipe flexibility is provided to account for shell rotation and deflections of the shell when filled.

7.2.3 *Flexibility.* Sufficient piping flexibility to accommodate seismic movements and settlement in the piping system shall be provided to protect the connections.

Sec. 7.3 Overflow

The tank shall be equipped with an overflow of the type and size specified. If a stub overflow is specified, it shall project at least 12 in. (300 mm) beyond the tank shell. If an overflow to ground is specified, it shall be brought down the outside of the tank shell and supported at proper intervals with suitable brackets. The overflow to the ground shall discharge over a drainage inlet structure or a splash block. It shall terminate at the top in a weir box or other suitable intake. The overflow pipe and intake shall have a capacity at least equal to the pumping rate as specified, with a water level not more than 6 in. (150 mm) above the weir. Unless otherwise specified, the overflow pipe shall be fitted with an elbow, screened flap valve or duckbill, and terminate 24 in. above ground. If carbon steel is specified, the overflow pipe shall have screwed or welded connections if it is smaller than 4 in. (100 mm) in diameter, or flanged or welded connections if it is 4 in. (100 mm) in diameter or larger. The maximum flow rate shall be specified, in gallons per minute, for which the overflow shall be designed. Internal overflows are not recommended but may be provided if specified. The internal overflow pipe shall have a minimum thickness of 0.25 in. (6.4 mm).

Sec. 7.4 Ladders

7.4.1 *Outside tank ladder.* An exterior ladder shall be provided from the bottom of the shell, or other specified level, to the top of the shell or roof at the specified orientation. The minimum clear width of step surface for rungs shall be 16 in. (400 mm), and rungs shall be equally spaced no more than 12 in. (305 mm) on center. The perpendicular distance from the centerline of the rungs to the tank wall shall not be less than 7 in. (180 mm). Rung size shall not be less than ¾ in. (19 mm) in diameter, or equivalent section. The maximum spacing of supports attaching the ladder to the tank shall not exceed 10 ft (3 m). The design loads shall be concentrated at such a point or points as will cause the maximum stress in the structural ladder member being considered. Side rails may be of any shape having

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section properties adequate to support the design loads and providing a means of securely fastening each rung to the side rail so as to lock each rung to the side rails.

7.4.2 *Inside tank ladder.* Inside tank ladders are not recommended. If an inside ladder is specified, it shall comply with the requirements of Sec. 7.4.1.

7.4.3 *Access to roof hatches and vents.* Unless otherwise specified, access to roof hatches and vents shall be provided. Such access shall be reached from the outside tank ladder according to the following:

1. For slopes 5 in 12 or greater, a ladder or stairway shall be provided.
2. Slopes less than 5 in 12 and greater than 2 in 12 shall be provided with a single handrail and nonskid walkway.
3. Slopes 2 in 12 or less do not require a handrail or nonskid surface.

7.4.4 *Minimum requirements.* Minimum requirements for ladders, cages, hatches, platforms, guardrails, stairs, and other safety components can be found in OSHA 29 CFR Part 1910, "Occupational Safety and Health Standards," General Industry Standards.

Sec. 7.5 Safety Devices

Requirements for ladder cages, rest platforms, guardrails, and other safety devices that exceed the requirements found in OSHA 29 CFR Part 1910 shall be specified.

Sec. 7.6 Roof Openings

7.6.1 *Above maximum water level.* A roof opening with a hinged cover and hasp for locking shall be provided above the MWL. The opening shall have a clear dimension of at least 24 in. (610 mm) in one direction and 15 in. (380 mm) in the other direction. The opening shall have either (1) a tight-fitting cover with a curb at least 4 in. (100 mm) in height, and the cover shall have a downward overlap of at least 2 in. (50 mm), or (2) a gasketed weathertight cover, in lieu of the 4-in. (100-mm) curb and 2-in. (50-mm) overlap. The roof opening may be omitted when a combined screened vent-manhole cover equipped with locking hasp is specified at the roof center opening.

7.6.2 *Roof center.* An opening, with a removable cover, having an opening dimension or diameter of at least 20 in. (510 mm) and 4 in. (100 mm) minimum height neck, shall be provided at, or near, the center of the tank. In lieu of the 4-in. (100-mm) neck, a gasketed, weathertight cover or a combined screened vent-manhole is acceptable.

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Sec. 7.7 Vent

If the tank roof is of tight construction, a suitable vent shall be provided above the MWL. The vent shall have a capacity to pass air so that at the maximum possible rate of the water, either entering or leaving the tank, excessive pressure will not develop. The overflow pipe shall not be considered a tank vent. The vent shall be designed and constructed to prevent the ingress of birds or animals.

7.7.1 *Location.* Even if more than one vent is required, one tank vent shall always be located near the center of the roof.

7.7.2 *Screening.* When insect screening is specified, a pressure-vacuum screened vent or a separate pressure-vacuum relief mechanism shall be provided that will operate in the event that the screens frost over or become clogged with foreign material. The screens or relief mechanism shall not be damaged by the occurrence and shall return automatically to the operating position after the clogging is cleared.

Sec. 7.8 Galvanic Corrosion

Dissimilar metals (e.g., stainless steel, copper, brass, etc.) more noble than carbon steel and installed inside the tank below the MWL shall be electrically isolated from the carbon steel tank components to which they are attached. This isolation is required even if the attachment point of the dissimilar metals to the tank is above the maximum water level.

Sec. 7.9 Cathodic Protection

If a cathodic protection system is provided, the criteria for design and the system installation shall comply with ANSI/AWWA D104 or ANSI/AWWA D106, as applicable to the type of system. On completion of tank construction, electrical continuity among all tank components in contact with water shall be verified.

SECTION 8: WELDING

Sec. 8.1 General

The field assembly of all major tank elements such as vertical, horizontal, shell-to-roof, and shell-to-bottom plates or sheets shall be by bolting. Welding shall be limited to the shop fabrication of nozzles, vents, manways, connections, and subassemblies. Field welding may only be performed when approved in accordance

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with the manufacturer's instructions and welding procedure specifications. Qualification of welding procedures and welders for shop and field welding shall be in accordance to applicable AWS codes referenced in Section 2.

Sec. 8.2 Welds

All welds shall be made according to the minimum requirements of AWS D1.1/D1.1M for carbon steel, AWS D1.6/D1.6M for stainless steel, and AWS D1.2/D1.2M for aluminum. If specified, a certification that all welds were made by AWS-qualified welders and inspected according to AWS standards shall be provided.

8.2.1 *Peening.* Peening of weld layers may be used to prevent undue distortion. Surface layers shall not be peened. Peening shall be performed with light blows from a power hammer using a blunt-nosed tool.

8.2.2 *Contour.* All welds that are to be grit-blasted before coating should be rough ground to remove any high points before grit blasting. All craters shall be filled to the full cross section of the weld.

8.2.3 *Reinforcement.* The reinforcement of butt-joint welds shall be as small as practicable, preferably not more than 1⁄16 in. (1.6 mm). In no case shall the face of the weld lie below the surface of the plates being joined.

8.2.4 *Gouging.* Gouging at the root of welds and gouging of welds to remove defects may be performed by arc or oxygen gouging.

8.2.5 *Cleaning between beads.* Each bead of multiple-pass weld shall be cleaned of slag and other loose deposits before the next bead is applied, as well as cleaning the final pass before inspection.

8.2.6 *Visual inspection.* All welds shall be visually inspected by an individual who by training or experience, or both, in metals fabrication, inspection, and testing is competent to perform the visual inspection. Any of the following weld defects identified by visual inspection shall be cause for rejection, and the deficient weld shall be repaired or replaced:

1. Any crack, regardless of size or location.
2. Lack of fusion between adjacent layers of weld metal or between weld metal and base metal.
3. Unfilled craters.
4. Overlap resulting from the protrusion of weld metal beyond the weld toe or weld root.
5. Weld size less than specified (insufficient throat or leg).
6. Butt joint reinforcement in excess of

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a. 1/16 in. (1.6 mm) for plates not greater than 5/16 in. (7.9 mm).
b. 3/32 in. (2.4 mm) for plates over 5/16 in. (7.9 mm).

7. Fillet weld convexity in excess of
   a. 1/16 in. (1.6 mm) for width of individual bead or weld face not over 5/16 in. (7.9 mm).
   b. 1/8 in. (3.2 mm) for width of individual bead or weld face over 5/16 in. (7.9 mm), but less than 1 in. (25.4 mm).
   c. 3/16 in. (4.8 mm) for width of individual bead or weld face 1 in. (25.4 mm) or over.
8. Undercut in excess of the limits given in Sec. 8.4.
9. Porosity:
   a. Any visible porosity in butt joints subject to primary stress.
   b. In all other joints, the sum of visible porosity greater than 1/32 in. (0.8 mm) in diameter shall not exceed 3/8 in. (9.5 mm) in any linear inch of weld and shall not exceed 3/4 in. (19.0 mm) in any 12-in. (305-mm) length of weld.
10. Plate misalignment in excess of tolerances noted on detail drawings.

Sec. 8.3 Preparation of Surfaces to Be Welded

Surfaces to be welded shall be free from loose scale, slag, heavy rust, grease, paint, and any other foreign material except tightly adherent mill scale. A light film of weldable rust-preventive coating or compound may be disregarded. Such surfaces shall also be smooth, uniform, and free from fins, tears, and other defects that adversely affect proper welding. A fine film of rust adhering to cut or sheared edges after wire brushing need not be removed.

Sec. 8.4 Undercuts and Penetration of Welds

Welds shall be examined visually for compliance with the following:

8.4.1 *Butt and lap joints subject to primary stress.* For butt and lap joints subject to primary stress caused by weight or pressure of tank contents, there shall be complete joint penetration, complete fusion, and no undercutting.

8.4.2 *Butt joint subject to secondary stress.* For butt joints subject to secondary stress, there shall be complete joint penetration, complete fusion, and no undercutting.

8.4.3 *Lap joints subject to secondary stress.* For lap joints subject to secondary stress, the maximum undercut permitted shall be 12½ percent of the thinnest sheet measured at each edge of the weld, but in no case greater than 1/32 in. (0.8 mm).

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Sec. 8.5 Cleaning of Welds

Weld scale or slag, spatter, burrs, and other sharp or rough projections shall be removed in a manner that will leave the surface suitable for any subsequent cleaning and coating operation. Weld seams need not be chipped or ground, provided they may be satisfactorily cleaned and coated, when applicable.

SECTION 9: SHOP FABRICATION

Sec. 9.1 Straightening

Any required straightening of material shall be done by methods that will not damage the steel. Minor cold straightening is permitted. Cold straightening may be done by hammering or, preferably, by rolling or pressing. Unless otherwise noted, heat may be used in straightening more severe deformations.

Sec. 9.2 Finish of Plate and Sheet Edges

The plate and sheet edges to be bolted or welded may be universal mill edges or they may be prepared by shearing, machining, or chipping, or by mechanically guided laser, water jet, or oxygen or plasma arc cutting.

9.2.1 *Oxygen or plasma arc cutting.* When edges of plates or sheets are oxygen or plasma arc cut, the surface obtained shall be uniform and smooth and shall be cleaned of slag accumulations. All cutting shall follow closely the lines prescribed.

9.2.2 *Shearing.* Shearing may be used for material ½ in. (13 mm) or less in thickness to be butt-welded, and for all thicknesses of material to be joined by bolted or welded lap joints.

Sec. 9.3 Rolling

Plates and sheets shall be cold-rolled or pressed to suit the curvature of the tank and the erection procedure.

Sec. 9.4 Forming

9.4.1 *Double-curved forming.* Plates and sheets that are curved in two directions may be pressed or rolled either cold or hot.

9.4.2 *Bend radii.* Minimum radii for cold-forming plates and sheets shall conform to suggested minimum radii in ASTM A6 or ASTM A568, whichever is applicable to the material grade being formed, including applicable appendixes.

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Sec. 9.5 Manufacturing Tolerances

9.5.1 *Tanks with horizontally flanged shell joints.* Parts fabricated and punched for tanks with horizontally flanged shell joints shall comply with the tolerances of API SPEC 12B, where applicable.

9.5.2 *Tanks with horizontally lapped shell joints.* The tolerance on bolt hole spacing for tanks with horizontally lapped shell joints shall be ±1⁄32 in. (±0.8 mm) between any two holes, measured in the flat before forming.

Sec. 9.6 Coatings

Bolted tanks are supplied with factory-applied coatings (refer to Section 12 for coatings).

Sec. 9.7 Shipping

All material shall be loaded, transported to the site, unloaded, and stored in such a manner as to prevent damage.

SECTION 10: ERECTION

Sec. 10.1 General

The tank shall be erected according to instructions provided with the tank. Instructions shall include, but not be limited to, details on bolting, placement of gaskets and/or sealants, coating repair procedures, and assembly procedures.

Sec. 10.2 Bolting

All bolts, nuts, and washers shall be located and installed in accordance with the erection instructions. Only hardware that is supplied with the tank shall be used.

Sec. 10.3 Gasketing and Sealants

Gasketing and/or sealants shall be supplied with the tank and installed between all joints in compliance with the erection instructions. Sealants shall be applied in accordance with the sealant manufacturer's instructions. Care shall be exercised in properly locating and installing all gaskets and/or sealants.

Sec. 10.4 Coating Repair

The coating shall be visually inspected and any damage to the factory-applied coatings shall be repaired in strict compliance with the erection instructions (see Sec. 12.2).

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Sec. 10.5 Cleanup

On completion of the erection, all rubbish and other unsightly material caused by the operations shall be properly disposed of, and the premises left in as good a condition as found at the start of the tank erection.

SECTION 11: INSPECTION AND TESTING

Sec. 11.1 Shop Inspection

11.1.1 *Shop inspection.* A shop inspection by a commercial inspection agency may be specified. Shop inspection shall at a minimum consist of a visual inspection of the fabricating practices, coating system application, and operations to determine compliance with this standard.

11.1.2 *Mill-test reports.* When specified, copies of certified mill-test reports shall be provided.

11.1.3 *Coating thickness test data.* When specified, certified test data on the coating thickness shall be provided.

Sec. 11.2 Testing

11.2.1 *Leak testing of the bottom.* When specified by the purchaser, the erector, before filling the tank with water, shall test the joints of the steel bottom using the vacuum method.

11.2.2 *Hydro test.* The tank shall be hydro-tested after erection by filling the tank with water to the MWL.

11.2.3 *Repair of leaks.* Any leaks in the tank below the MWL shall be repaired in accordance with the tank manufacturer's recommendations.

11.2.4 *Holiday testing.* When specified, independent field holiday detection testing of the completed tank shall be performed on the interior coated surfaces below the MWL in accordance with Sec. 12.9.

Sec. 11.3 Disinfecting

If specified, the tank shall be disinfected after the final test (see ANSI/AWWA C652). The tank may then be filled with potable water and placed into service.

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SECTION 12: COATINGS

Sec. 12.1 General

Bolted tanks are manufactured by several tank manufacturers and coated in the manufacturers' own facilities and shipped worldwide. The following generic systems are representative of those in general use. Equivalent generic systems, for which documentation consisting of test data, service history, and toxicological information as provided by the tank manufacturer, shall be considered for use in storage tanks under the provisions of this standard. For tanks storing potable water, immersed coatings shall have been tested and certified for potable water contact in accordance with NSF/ANSI 61 when required.

Sec. 12.2 Coating Repair

Tank erection instructions and procedures shall be strictly followed for field repair and touch-up of damaged coatings.

Sec. 12.3 Galvanized Coatings

When hot-dip galvanized coatings are specified, zinc metal suitable for immersion in drinking water shall be applied to the tank parts after fabrication in accordance with the recommended practice of the American Galvanizers Association* in compliance with ASTM A123 and ASTM A153.

Sec. 12.4 Glass Coatings

When glass fused-to-steel coatings are specified, the coatings shall be applied according to the tank manufacturer's specific procedure. Glass coatings are to comply with the following:

12.4.1 *Surface preparation.* The steel shall be cleaned of all oils and lubricants. Mill scale and rust must be removed from the steel surface by grit-blasting in accordance with SSPC-SP 10/NACE No. 2 or by pickling in compliance with SSPC-SP 8.

12.4.2 *Coatings.*

12.4.2.1 The steel is to be primed with applications of catalytic nickel oxide when tanks are fabricated from hot-rolled steel.

12.4.2.2 Glass coatings shall be applied by wet spraying, flow coating, dipping, or electrophoretic deposition. The coating thickness shall be between 6 mil (150 μm) and 19 mil (480 μm).

* American Galvanizers Association, 6881 South Holly Circle, Suite 108, Centennial, CO 80112.

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12.4.2.3 The glass coating must be cured or fused to the steel by firing. The temperature should be above 1,200°F (650°C) and preferably in the range of 1,450–1,600°F (790–870°C).

12.4.3 *Inspection.* Interior and exterior coated surfaces shall be inspected for any visible defect or holiday. Interior coating inspection shall include a holiday detection test (Sec. 12.9). Any coating defect shall be repaired and shall pass inspection before shipment.

Sec. 12.5 Thermoset Liquid Suspension Coatings

When thermoset liquid suspension epoxies are specified, the coatings shall be applied according to the tank manufacturer's specific procedure. Coatings are to comply with the following.

12.5.1 *Surface preparation.* Surface preparation shall comply with the following:

12.5.1.1 The steel shall be thoroughly cleaned by a wash-rinse followed immediately by hot air drying.

12.5.1.2 The steel shall then be grit-blasted on both sides in accordance with SSPC-SP 10/NACE No. 2. The surface profile shall be a minimum of 1 mil (25 μm).

12.5.2 *Coatings.* The coatings are to be applied in compliance with the following:

12.5.2.1 Within 30 min of blast cleaning, the interior surfaces of the tank shall receive one coat of amine-cured thermoset epoxy in strict accordance with the coating manufacturer's recommendations.

12.5.2.2 The exterior surfaces of the tank shall receive one coat of epoxy primer or equivalent.

12.5.2.3 The interior and exterior coatings shall be oven-heated until the coats have a tacky finish, with partial thermal cross-linking.

12.5.2.4 The interior surfaces of the tank shall receive a second coat of amine-cured epoxy to provide a total of 5-mil (127-μm) minimum dry film thickness.

12.5.2.5 The exterior surfaces of the tank shall receive a finish coat of acrylic baking enamel and be thermally cured. Minimum dry film thickness shall be a total of 3 mil (76 μm).

12.5.2.6 The interior and exterior finish coats shall be oven-heated at 425–525°F (220–275°C) for a minimum of 10 min to completely thermal crosslink both thermoset coatings.

12.5.2.7 Alternate exterior coating systems may be specified.

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12.5.3 *Inspection.* Interior and exterior coated surfaces shall be inspected for any visible defect or holiday and have their coating thickness verified by a nondestructive mil-thickness test (Mikrotest or equal). Interior coating inspection shall include a holiday detection test (Sec. 12.9) and solvent rub test, in accordance with ASTM D5402 for organic coatings and ASTM D4752 for ethyl silicate (inorganic) zinc-rich primers. Any coating defect shall be repaired and shall pass inspection before shipment.

Sec. 12.6 Thermoset Powder Coatings

When thermoset powder coatings are specified, the coatings shall be applied according to the tank manufacturer's specific procedure. Thermoset powder coatings are to comply with the following:

12.6.1 *Surface preparation.* The steel shall be steel-grit-blasted on all sides in accordance with SSPC-SP 10/NACE No. 2.

12.6.2 *Application.* The coating is to be applied in compliance with the following:

12.6.2.1 Within 30 min. of blast cleaning, the interior and exterior surfaces shall be dry-powder coated by electrostatic application with a powder coating.

12.6.2.2 The dry powder shall be deposited at a rate to yield 5-mil (127-μm) minimum dry film thickness interior and 3-mil (76-μm) minimum dry film thickness exterior.

12.6.2.3 The surfaces shall be oven-cured in accordance with the coating manufacturer's recommendations.

12.6.3 *Inspection.* Interior and exterior coated surfaces shall be inspected for any visible defect or holiday and have their coating thickness verified by a nondestructive mil-thickness test (Mikrotest or equal). Interior coating inspection shall include a holiday detection test (Sec. 12.9) and solvent rub test in accordance with ASTM D5402 for organic coatings and ASTM D4752 for ethyl silicate (inorganic) zinc-rich primers. Any coating defect shall be repaired and shall pass inspection before shipment.

Sec. 12.7 Marking

All of the tank components shall be given a piece mark number, or other identifier, for ease of assembly.

Sec. 12.8 Protection

All coated parts shall be protected from damage during shipment.

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Sec. 12.9 Holiday Testing

All holiday tests shall be nondestructive and shall use an electric DC meter and a wet sponge holiday detector operated by a trained technician in accordance with NACE SP0188 and ASTM D5162. The maximum voltage of the meter shall not exceed 67.5 volts. The sponge shall be dipped in plain tap water as required to keep it uniformly damp, not soaked or dry. Unless specifically required by the testing equipment manufacturer, no "conductive" or "wetting" additives shall be used. Refer to the recommendations of the testing equipment manufacturer for proper setting, testing, and operation of the equipment.

SECTION 13: FOUNDATION DESIGN AND CONSTRUCTION

Sec. 13.1 General Requirements

The foundations are important because unequal settlement changes the distribution of stresses in the structure and may cause leakage or buckling of the tank.

13.1.1 *Foundation plans.* Construction drawings of the foundation shall be furnished. The type of foundation and foundation depth shall be based on a properly conducted soil investigation. When specified or required by design, anchor bolts shall be provided.

13.1.2 *Foundation installation.* The earth around the foundation shall be regraded sufficiently to permit efficient work during tank erection and to prevent ponding of water in the foundation area. The tops of the foundations shall be accurately located at the proper elevation.

13.1.3 *Water load.* Water load as defined in Sec. 5.2.3 shall be considered as live load as defined by ACI 318 (see Sec. 13.6). The appropriate factors for all live loads shall be used in foundation design.

Sec. 13.2 Geotechnical

The design soil-bearing pressure shall be specified and shall include an appropriate factor of safety (Sec. 13.3) that is based on a properly conducted soil investigation (Sec. 13.2.1). In no case shall the specified bearing pressure exceed that which would cause intolerable settlements and impair the structural integrity of the tank.

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13.2.1 *Soil investigation.* A soil investigation shall be performed to determine the following:

1. The presence or absence of rock, old excavation, or fill.
2. Whether the site is a suitable place on which to build the structure.
3. The classification of soil strata, after appropriate sampling.
4. The type of foundation that will be required at the site.
5. The elevation of groundwater and whether dewatering is required.
6. The bearing capacity of the soil and the depth at which the foundation must be founded.
7. Whether a deep foundation will be required, and the required length of piles, caissons, piers, and so on.
8. The elevations of the existing grade and other topographical features that may affect the foundation design or construction.
9. The homogeneity and compressibility of the soils across the tank site, and expected magnitude of uniform and differential settlements.
10. Specification of type, quality, and minimum compaction of sub-base materials required for support of the tank and contents. See foreword, Sec. II.D.
11. The shear capacity of the soil, including minimum allowable width of ringwall, if applicable.
12. Site Class in accordance with Sec. 14.2.3.

Sec. 13.3 Safety Factors

The following minimum safety factors shall be used in determining the allowable soil-bearing pressure. The ultimate bearing capacity should be based on sound principles of geotechnical engineering and a properly conducted soil investigation. See foreword, Sec. II.D.

1. A safety factor of 3 shall be provided, based on calculated ultimate bearing capacity for all direct static loads.
2. A safety factor of 2.25 shall be provided, based on calculated ultimate bearing capacity for all direct loads that include wind or seismic.

Sec. 13.4 Foundation Types

All steel-bottom tanks shall be supported on a concrete ringwall, concrete slab, or structurally compacted granular berm, with or without concrete or steel retainer rings. All concrete-bottom tanks shall consist of a base-setting ring embedded in concrete. The type of foundation shall be specified. The top of the foundation shall be a minimum of 6 in. (152 mm) above the finished grade, unless otherwise

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specified. Tanks that require anchor bolts shall only be supported on a concrete ringwall or a concrete slab. Specifications for the preparation of sub-base material shall be provided, if specified (foreword, Sec. II.D).

The tank foundation shall be one of the following types.

13.4.1 *Type 1. Steel-bottom tanks supported on ringwalls.* A sand or fine stone cushion at least 3-in. (76-mm) thick shall be provided above the earthen interior under the tank bottom. A 1-in. (25-mm) minimum space between the tank bottom and the top of the ringwall shall be filled with a high-strength nonshrink grout. The grout shall fill the entire space beneath the tank from the outside edge of the tank bottom to the cushion. In no case shall the width of grout placed under the tank bottom be less than 6 in. (150 mm). The top of the foundation shall be thoroughly cleaned and wetted with water before grout is placed.

When the tank is a mechanically anchored tank or when the foundation under the shell does not meet the tolerances of Sec. 13.6.1, the tank shall be supported by grout. For self-anchored tanks where the foundation under the shell meets the tolerances of Sec. 13.6.1, the shell may be supported with ½-in. (13-mm) thick cane-fiber joint filler meeting the requirements of ASTM D1751. When grouted: (a) a 1-in. (25-mm) minimum space between the tank bottom and the top of the concrete ringwall shall be filled with a high-strength nonshrink grout; (b) the grout shall fill the entire space beneath the tank from the outside edge of the tank bottom to the cushion; (c) in no case shall the width of grout placed under the tank bottom be less than 6 in. (150 mm); and (d) the top of the foundation shall be cleaned and thoroughly wetted with water before grout is placed.

13.4.2 *Type 2. Steel-bottom tanks supported on concrete slabs.* A sand or fine stone cushion not less than 1-in. (25-mm) thick shall be provided between the flat bottom and the concrete slab foundation. In lieu of a cushion, the bottom may be supported on a minimum ½-in. (13-mm) thick cane-fiber joint filler meeting the requirements of ASTM D1751. When the tank is a mechanically anchored tank or when the foundation under the shell does not meet the tolerances of Sec. 13.6.1, the tank shall be supported by grout. For self-anchored tanks where the foundation under the shell meets the tolerances of Sec. 13.6.1, the shell may be supported with ½-in. (13-mm) thick cane-fiber joint filler meeting the requirements of ASTM D1751. When grouted: (a) a 1-in. (25-mm) minimum space between the tank bottom and the top of the concrete ringwall shall be filled with a high-strength nonshrink grout; (b) the grout shall fill the entire space beneath the tank from the outside edge of the tank bottom to the cushion; (c) in no case shall the width of

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grout placed under the tank bottom be less than 6 in. (150 mm); and (d) the top of the foundation shall be cleaned and thoroughly wetted with water before grout is placed.

13.4.3 *Type 3. Steel-bottom tanks within concrete retainer rings.* Tanks may be placed on a cushion within a concrete retainer ring. The cushion shall consist of a minimum of 6 in. (150 mm) of sand or fine stone. The inside of the ringwall is to be a minimum of 12 in. (300 mm) outside the bottom plates of the tank. Adequate provisions for drainage inside the ringwall must be made.

13.4.4 *Type 4. Steel-bottom tanks supported on granular berms.* The berm shall be well-graded stone or gravel. The berm shall extend a minimum of 3 ft (0.9 m) beyond the tank shell and from there have a maximum slope of 1:1.5. The berm under the shell shall be level within ±⅛ in. (±3 mm) in any 10 ft (3 m) of circumference and within ±½ in. (±13 mm) in the total circumference. Adequate protection shall be provided to ensure against foundation washout.

13.4.5 *Type 5. Steel-bottom tanks supported on granular berms with steel retainer rings.* The berm shall be well-graded stone or gravel. The berm shall extend to the retainer ring. The size and coating of the steel retainer ring shall be specified and shall be a minimum of 12 in. (300 mm) from the shell or a sufficient distance to ensure berm stability under the shell in the event that the steel retainer ring is removed. The berm under the shell shall be level within ±⅛ in. (±3 mm) in any 10 ft (3 m) of circumference and within ±½ in. (±13 mm) in the total circumference.

13.4.6 *Type 6. Concrete-bottom tanks with embedded steel base setting ring.* The base-setting ring shall be properly assembled in strict accordance with the construction drawings and rigidly supported and attached to a concrete ringwall footing before placement of concrete for the curb and tank bottom (see Sec. 13.6.4 for tolerances). A greater depth may be required for design, but the base-setting ring shall be embedded in concrete at least 6 in. (150 mm). A minimum distance of 3 in. (76 mm) between the top of the footing and the bottom of the base-setting ring shall be provided. Unless a greater width is required for design, the exterior curb shall have a width of 8 in. (200 mm), and its finished top shall coincide ±1 in. (±25 mm) with the finished top of the concrete bottom. A minimum of one elastomer water stop, bentonite strip, or other suitable water-sealing material shall be installed on the interior surface of the base-setting ring, completely around the entire circumference, before placement of concrete for the curb and tank bottom. The top of the sealing material shall be a minimum distance of 2 in. (50 mm) below

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the finished top of the concrete bottom. Concrete shall be reinforced and designed in accordance with ACI 318. Additional reinforcing steel shall be installed around the base-setting ring, as required, to control shrinkage and resist horizontal loads, in accordance with the construction drawings (Sec. 1.2).

Sec. 13.5 Foundation Design Details

13.5.1 *Height above ground.* The tops of the concrete foundations shall be a minimum of 6 in. (150 mm) above the adjacent grade, unless otherwise specified.

13.5.2 *Minimum depth of foundations.* The extreme frost penetration depths in Figure 1 shall be the minimum depth of foundation below the finished grade. However, the minimum depth below finished grade shall not be less than 12 in. (300 mm). Foundation depth shall be increased in localities where soil or other factors are favorable to deep frost penetration and may be reduced for foundations resting on rock. Consult local records for the extreme frost penetration in the circled area of Figure 1. Uplift or soil-bearing requirements may dictate greater depths.

13.5.3 *Size of top.* The tops of foundations shall project at least 3 in. (76 mm) beyond the tank sidewall, or greater if required by design. In base-setting

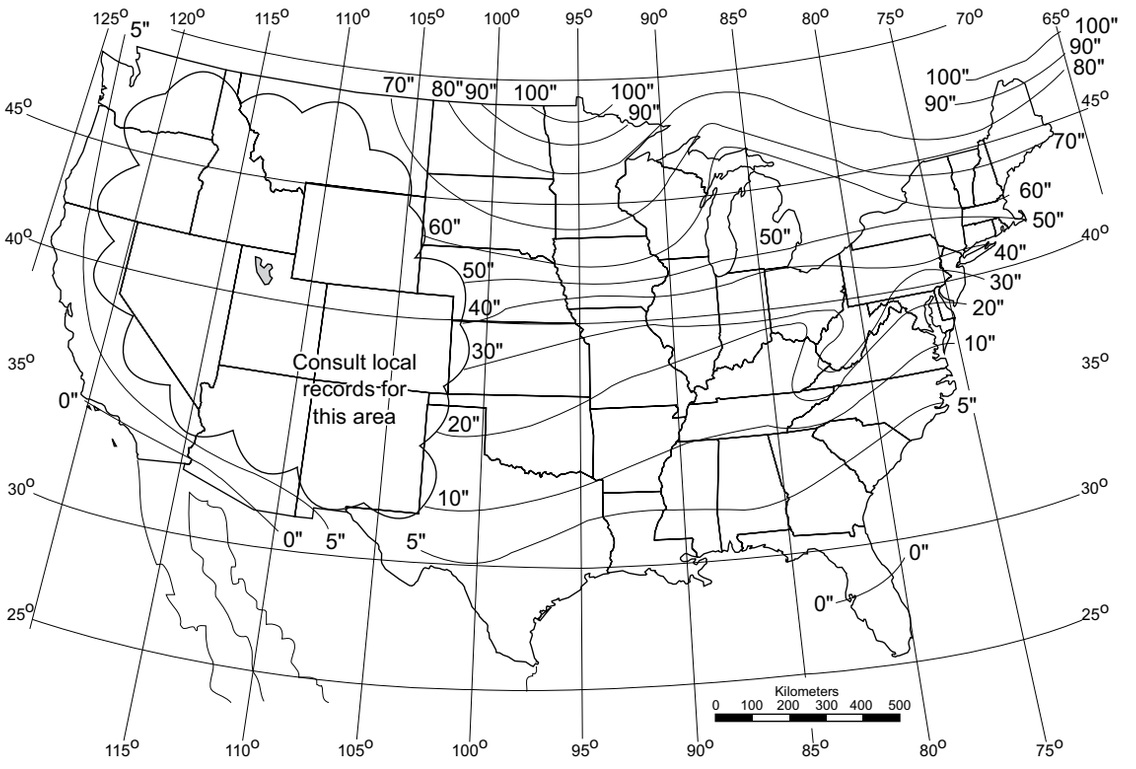


Figure 1 Extreme frost penetration—inches (based on state averages)

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ring applications, the top of the foundation should project a minimum of 8 in. (200 mm) beyond the tank sidewall, or greater if required by design. When anchor bolts are used, the foundations shall project 9 in. (230 mm) beyond the tank sidewall, or greater as required by anchor bolt design (Sec. 5.9). The top corners of the foundation shall be either neatly rounded or finished with a suitable bevel.

13.5.4 *Deep foundations.* If a deep foundation is required using piles, caissons, piers, etc., the type of supports, depth below existing grade, and design capacities for all loads and load combinations—including live, dead, weight of soil above the footing, wind, and seismic—shall be specified.

Sec. 13.6 Concrete Design, Materials, and Construction

The design of the concrete foundations, the specifications for the cement and aggregate, and the mixing and placing of the aggregate shall be in accordance with ACI 318, except as may be modified in this section and the following subsections. Concrete work shall conform to all requirements of ACI 301, unless otherwise specified.

13.6.1 *Tolerances on concrete foundations.* Ringwalls and slabs, after grouting or before placing the cane-fiber joint filler, shall be level within ±⅛ in. (±3 mm) in any 30-ft (9-m) circumference under the shell. The levelness on the circumference shall not vary by more than ±¼ in. (±6 mm) from an established plane. The tolerance on poured concrete before grouting shall be ±1 in. (±25 mm).

13.6.2 *Finish.* The top portions of foundations, to a level 6 in. (150 mm) below the proposed ground level, shall be finished to a smooth form finish in compliance with ACI 301. The top corners of the foundation shall be either neatly rounded or finished with a suitable bevel. Any small holes may be troweled over with mortar as soon as possible after the forms are removed.

13.6.3 *Tolerances on anchor bolts.* Anchor bolt location, projection, and embedment tolerance shall be ±¼ in. (±6 mm). Anchor bolt plumbness tolerance shall be ±3° from vertical.

13.6.4 *Tolerances on base-setting rings.* Base-setting rings shall be level ±1⁄16 in. (±1.6 mm) and concentric ±¼ in. (±6 mm). It is extremely important that the base-setting ring be assembled and installed in strict accordance with these tolerances and the tank manufacturer's instructions.

Sec. 13.7 Backfill

For tanks with ringwall foundation, all topsoil, organic material, and undesirable material within the ringwall shall be removed and replaced with a

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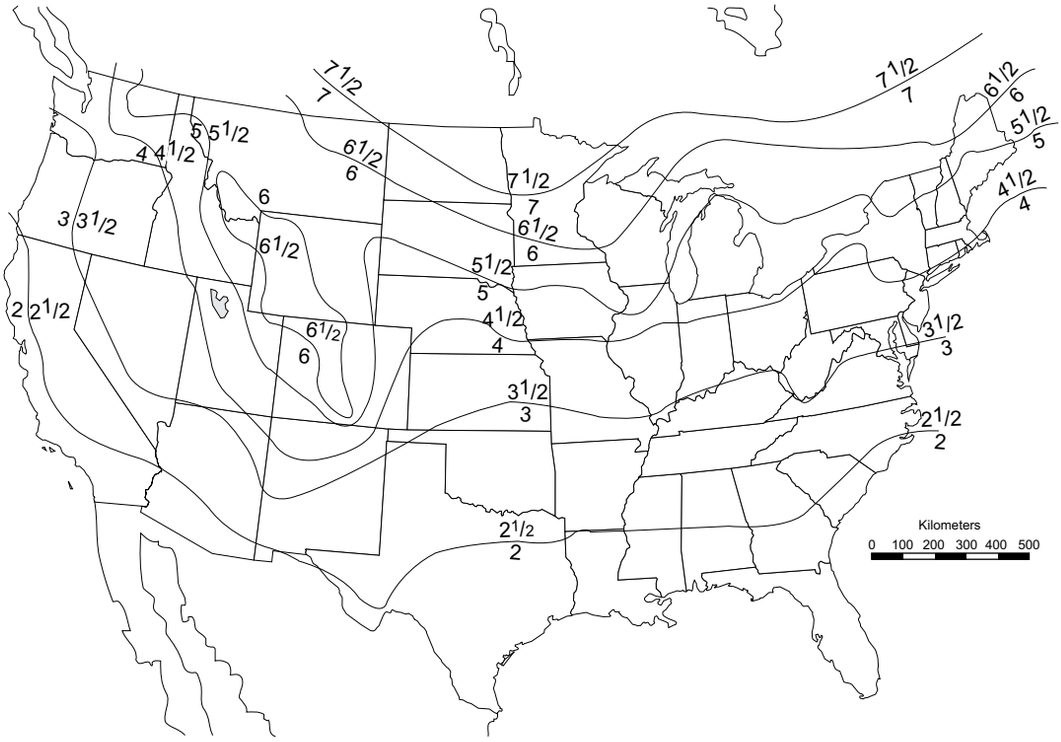


Figure 2 Recommended depth of pipe cover—feet above top of pipe

controlled, load-bearing backfill. The natural soils and load-bearing backfill within the ringwall shall be capable of supporting the tank bottom without general or localized settlement that may damage the tank.

13.7.1 *Material and compaction.* Load-bearing backfill shall be suitable nonfrozen material placed and compacted in uniform horizontal lifts to the degree of compaction required by the foundation design. The water load and ringwall height shall be considered in determining the required degree of compaction.

13.7.2 *Pipe cover.* Pipe cover shall be provided in compliance with Figure 2 unless local conditions dictate that more or less cover should be used.

SECTION 14: SEISMIC DESIGN

Sec. 14.1 General

14.1.1 *Scope.* The design earthquake ground motion in this standard is derived from ASCE 7 and is based on a maximum considered earthquake ground motion defined as the motion due to an event with a 2 percent probability of

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exceedance within a 50-year period (recurrence interval of approximately 2,500 years). Application of these provisions, as written, is deemed to meet the intent and requirements of ASCE 7. Techniques for applying these provisions where regulatory requirements differ from ASCE 7 are provided in the commentary.

Tanks located where S_1 (Sec. 14.2.2) is less than or equal to 0.04 and S_S (Sec. 14.2.2) is less than or equal to 0.15 need not be designed for seismic forces. Where design for seismic forces is required by this standard, the design earthquake ground motion shall be determined using the general procedure (Sec. 14.2.6) or, when specified or required by this standard, the site-specific procedure (Sec. 14.2.7). Seismic design forces are reduced to service-level forces by the use of strength-level to service-level factor λ_E (refer to Sec. 14.2.8).

Alternative procedures that account for the effects of soil-structure interaction for mechanically anchored tanks are permitted in Sec. 14.2.9 and must meet specific criteria.

14.1.2 *Definitions.*

1. Active fault. A fault with an average historic slip rate of at least 1 mm per year and geologic evidence of seismic activity within Holocene time (i.e., past 11,000 years).

2. 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.

3. Convective component. The convective component represents the sloshing portion of the contents and is characterized by a long natural period.

4. Impulsive component. The impulsive component represents the portion of the contents that moves in unison with the shell.

5. Mechanical-anchoring. Utilizing anchor bolts or base-setting ring to anchor the tank to the foundation.

6. Risk-adjusted maximum considered earthquake (MCE_R). The most severe earthquake considered by ASCE 7 determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk.

7. Self-anchoring. Utilizing the self-weight of the tank and contents to resist overturning forces.

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Table 2 Seismic importance factor I_E

Risk Category*	Seismic Importance Factor I_E
II	1.00
III	1.25
IV	1.50

*Risk Category I was deleted. Refer to Sec. 5.2.1 for definitions of risk categories.

Sec. 14.2 Design Earthquake Ground Motion

14.2.1 *Seismic importance factor I_E.* The seismic importance factor I_E is based on the risk category (Sec. 5.2.1) and shall be in accordance with Table 2.

14.2.2 *Mapped acceleration parameters.* Mapped risk-adjusted maximum considered earthquake (MCE_R) spectral response accelerations, 5 percent damped, at 0.2-s period S_S and 1-s period S_1 shall be obtained from ASCE 7, Figures 22-1 through 22-6.

14.2.3 *Site class.* Site class accounts for the effect of local soil conditions on the ground motion and shall be based on the types of soil present and their engineering properties. The types of soil present and their engineering properties shall be established by a geotechnical investigation. The site shall be classified in accordance with ASCE 7 chapter 20. Site Class D shall be used when the soil properties are not known in sufficient detail to determine the site class, provided Site Class E or F soils are not present at the site.

A site response analysis that complies with Sec. 14.2.7 is required for sites classified as Site Class F, except as follows. For structures having fundamental periods of vibration equal to or less than 0.5 s, a site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with ASCE 7, Chapter 20, and the corresponding values of F_a and F_v determined in accordance with Sec. 14.2.4.

14.2.4 *Site coefficients F_a and F_v.* Short-period site coefficient F_a and long-period site coefficient F_v are used to modify mapped spectral response accelerations for 0.2-s and 1-s periods, respectively, for site classes other than B. Site coefficients F_a and F_v shall be in accordance with Tables 3 and 4, respectively.

14.2.5 *Response modification factors R_i and R_c.* The response modification factor accounts for damping, overstrength, and the ductility inherent in the tank at displacements great enough to surpass initial yield and approaching the ultimate load displacement of the tank. The response modification factor applied to the

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Table 3 Short-period site coefficient F_a

	Mapped Spectral Response Acceleration at 5 Percent Damping and 0.2-s Period*				
Site Class	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
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	†	†	†	†	†

* Use straight-line interpolation for intermediate values of S_S.
† Site-specific evaluation and procedure (Sec. 14.2.7) are required.

Table 4 Long-period site coefficient F_v

	Mapped Spectral Response Acceleration at 5 Percent Damping and 1-s Period*				
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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	†	†	†	†	†

* Use straight-line interpolation for intermediate values of S_1.
† Site-specific evaluation and procedure (Sec. 14.2.7) are required.

Table 5 Response modification factors R_i and R_c

	Response Modification Factor	
Tank Type	R_i (impulsive component)	R_c (convective component)
Mechanically-anchored	3.0	1.5
Self-anchored	2.5	1.5

impulsive component R_i and the response modification factor applied to the convective component R_c shall be in accordance with Table 5.

14.2.6 *Design response spectra—general procedure.*

14.2.6.1 General. The general procedure is based on the mapped MCE_R spectral response accelerations from Sec. 14.2.2 for an event with a 2 percent probability of exceedance within a 50-yr period.

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14.2.6.2 MCE_R spectral response acceleration. Mapped MCE_R spectral response accelerations from Sec. 14.2.2 shall be adjusted for site class using the following equations.

$$S_{MS} = F_a S_S \quad \text{(Eq 14-1)}$$

$$S_{M1} = F_v S_1 \quad \text{(Eq 14-2)}$$

where:

S_{MS} = MCE_R spectral response acceleration, 5 percent damped, at 0.2-s period and adjusted for site class effects, stated as a multiple (decimal) of g

S_{M1} = MCE_R spectral response acceleration, 5 percent damped, at 1-s period and adjusted for site class effects, stated as a multiple (decimal) of g

S_S = mapped MCE_R spectral response acceleration, 5 percent damped, at 0.2-s period for Site Class B from Sec. 14.2.2, stated as a multiple (decimal) of g

S_1 = mapped MCE_R spectral response acceleration, 5 percent damped, at 1-s period for Site Class B from Sec. 14.2.2, stated as a multiple (decimal) of g

F_a = short-period site coefficient from Table 3

F_v = long-period site coefficient from Table 4

g = acceleration caused by gravity in ft/s^2 (m/s^2)

14.2.6.3 Design response spectra. Design response spectra for impulsive and convective components shall be based on design earthquake spectral response accelerations, 5 percent damped, at 0.2-s period S_{DS} and 1-s period S_{D1}.

$$S_{DS} = US_{MS} \quad \text{(Eq 14-3)}$$

$$S_{D1} = US_{M1} \quad \text{(Eq 14-4)}$$

where:

S_{DS} = design earthquake spectral response acceleration, 5 percent damped, at 0.2-s period, stated as a multiple (decimal) of g

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S_{D1} = design earthquake spectral response acceleration, 5 percent damped, at 1-s period, stated as a multiple (decimal) of g

U = scaling factor to scale the MCE_R spectral response acceleration to the design earthquake spectral response acceleration

= ⅔

The other symbols have been previously defined in this section.

14.2.6.3.1 Design response spectrum for impulsive components. The design response spectrum for impulsive components shall be based on 5 percent damping and the following equations:

For $0 \le T_i \le T_S$: $$S_{ai} = S_{DS} \quad \text{(Eq 14-5)}$$

For $T_S < T_i \le T_L$: $$S_{ai} = \frac{S_{D1}}{T_i} \le S_{DS} \quad \text{(Eq 14-6)}$$

For $T_i > T_L$: $$S_{ai} = \frac{T_L S_{D1}}{T_i^2} \quad \text{(Eq 14-7)}$$

where:

S_{ai} = design spectral response acceleration for impulsive components, 5 percent damped, at the natural period of the structure T_i, stated as a multiple (decimal) of g.

T_i = natural period of the structure, in seconds

T_L = region-dependent transition period for longer-period ground motion, in seconds, from ASCE 7, Figures 22-12 through 22-16

T_S = $\frac{S_{D1}}{S_{DS}}$

The other symbols have been previously defined in this section.

14.2.6.3.2 Design response spectrum for the convective component. The design response spectrum for the convective component shall be based on 0.5 percent damping and the following equations:

For $T_c \le T_L$: $$S_{ac} = \frac{KS_{D1}}{T_c} \le S_{DS} \quad \text{(Eq 14-8)}$$

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For $T_c > T_L$: $$S_{ac} = \frac{KT_L S_{D1}}{T_c^2} \quad \text{(Eq 14-9)}$$

where:

S_{ac} = design spectral response acceleration for the convective component, 0.5 percent damped, at the first mode sloshing wave period T_c, stated as a multiple (decimal) of g

K = damping scaling factor

= 1.5 to convert spectrum from 5 percent damping to 0.5 percent damping

T_c = first mode sloshing wave period in seconds

The other symbols have been previously defined in this section.

14.2.7 *Design response spectra—site-specific procedure.*

14.2.7.1 General. The site-specific procedure only applies when specified or required by this standard. The site-specific procedure is required if the tank is located on Site Class F soils. When the site-specific procedure is specified or required by this standard, the site-specific evaluation geotechnical investigation and dynamic site response analyses shall be in accordance with Sec. 3.4 of FEMA 450.

14.2.7.2 Probabilistic MCE_R ground motion. The probabilistic MCE_R ground motion shall be represented by a 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance within a 50-yr period. The probabilistic MCE_R spectral response acceleration at any period S_{aM} shall be taken from that spectrum.

14.2.7.3 Deterministic MCE_R ground motion. The deterministic MCE_R ground motion shall be based on the dynamic site response analyses and shall be taken as 150 percent of the median 5 percent damped spectral response accelerations S_{aM} at all periods resulting from a characteristic earthquake on any known active fault within the region. The deterministic MCE_R ground motion response spectrum shall not be less than the corresponding ordinates of the response spectrum determined in Sec. 14.2.7.4.

14.2.7.4 Deterministic lower limit on MCE_R ground motion. The deterministic lower limit for maximum considered earthquake ground motion is defined by the response spectrum shown in Figure 3. Site coefficients F_a and F_v used to determine the deterministic limit shall be based on mapped spectral response accelerations at 0.2-s period S_S equal to $1.5g$ and 1-s period S_1 equal to $0.6g$.

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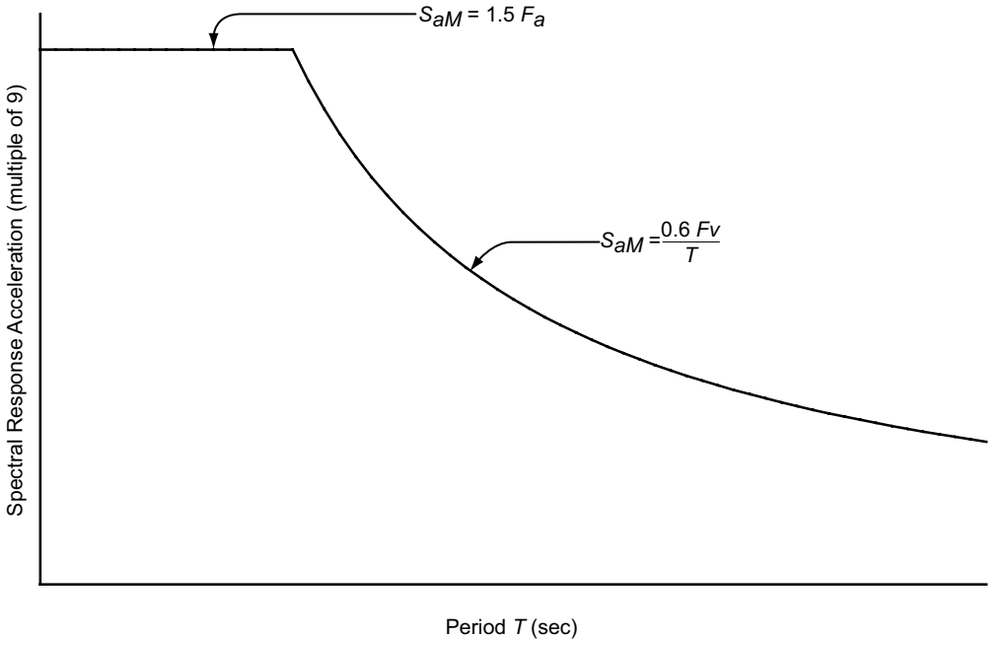


Figure 3 Deterministic lower limit for maximum considered earthquake ground motion

14.2.7.5 Site-specific MCE_R ground motion. The 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 maximum MCE_R ground motion (Sec. 14.2.7.2) and the deterministic MCE_R ground motion (Sec. 14.2.7.3).

14.2.7.6 Design response spectrum.

14.2.7.6.1 Design response spectrum for impulsive components. The design response spectrum for impulsive components S_{ai} shall be based on 5 percent damping and Eq 14-10 except as noted. The design spectral response acceleration by Eq 14-10 shall not be less than 80 percent of the design spectral response acceleration by the general procedure (Sec. 14.2.6). For sites classified as Site Class F requiring site-specific evaluations, the design spectral response acceleration at any period shall not be less than 80 percent of the design spectral response acceleration for Site Class E by the general procedure (Sec. 14.2.6).

$$S_{ai} = US_{aM} \qquad \text{(Eq 14-10)}$$

where:

S_{aM} = MCE_R spectral response acceleration, stated as a multiple (decimal) of g

The other symbols have been previously defined in this section.

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For tanks with H (distance from bottom of shell to MOL) to D (tank diameter) ratios equal to or less than 0.8, the design spectral response acceleration by Eq 14-10 may be limited to $(W_T / W_i)(\tan 30°)$ when the tanks are:

1. Self-anchored.

2. Mechanically anchored and are not otherwise prevented from sliding laterally at least 1 in. See Sec. 14.3.2.1 and Sec. 14.3.2.2.1 for definitions of W_T and W_i.

14.2.7.6.2 Design response spectrum for the convective component. The design response spectrum for the convective component shall be based on 0.5 percent damping and Eq 14-11. The design spectral response acceleration by Eq 14-11 shall not be less than 80 percent of the design spectral response acceleration by the general procedure (Sec. 14.2.6).

$$S_{ac} = UKS_{aM} \qquad \text{(Eq 14-11)}$$

The symbols have been previously defined in this section.

Alternatively, the design spectral response acceleration for the convective component S_{ac} may be taken from a 0.5 percent damped site-specific response spectrum based on the requirements of Sec. 14.2.7, except that the damping scaling factor K shall be set equal to 1.0.

14.2.8 *Horizontal design accelerations.*

14.2.8.1 For the general procedure, the impulsive design acceleration A_i is independent of T_i and S_{ai} shall be taken as S_{DS}. For the site-specific procedure, the impulsive design acceleration A_i shall be based on the design spectral response acceleration, 5 percent damped, S_{ai} for the natural period of the shell-fluid system. The convective design acceleration A_c shall be based on the spectral response acceleration, 0.5 percent damped, S_{ac} at the first mode sloshing wave period T_c. The first mode sloshing wave period shall be determined in accordance with Sec. 14.3.1. The design spectral response accelerations for impulsive and convective components shall be taken from design spectra determined by the general procedure (Sec. 14.2.6) or, when specified or required, the site-specific procedure (Sec. 14.2.7). The impulsive and convective design accelerations shall be determined by the following equations:

$$A_i = \frac{\lambda_E S_{ai} I_E}{R_i} \geq \frac{0.36 S_1 I_E}{R_i} \qquad \text{(Eq 14-12)}$$

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$$A_c = \frac{\lambda_E S_{ac} I_E}{R_c} \quad \text{(Eq 14-13)}$$

where:

A_i = impulsive design acceleration, stated as a multiple (decimal) of g
A_c = convective design acceleration, stated as a multiple (decimal) of g
R_c = response modification factor for the convective component from Table 5 for the type of structure
I_E = Seismic importance factor from Table 2
λ_E = Seismic load reduction factor to convert from strength level to service level
= 0.7

The other symbols have been previously defined in this section.

14.2.9 *Alternate procedures.* The effects of soil-structure interaction may be considered in accordance with ASCE 7 and the following limitations:

1. The tank shall be mechanically anchored to the foundation.

2. The tank shall be supported by a reinforced concrete foundation that is supported by soil or piles. Soil-structure interaction effects shall not be applied to tanks supported by granular berm foundations.

3. When soil-structure interaction is used, the procedure shall be similar to that given in ASCE 7 and the effective damping factor for the structure-foundation system shall not exceed 15 percent.

4. The base shear and overturning moment for the impulsive mode, including the effects of soil-structure interaction, shall not be less than 75 percent of the corresponding values without the effects of soil-structure interaction.

Sec. 14.3 Seismic Design Loads

14.3.1 *Natural periods.* The effective mass procedure considers two response modes of the tank and its contents: (1) the impulsive component is the high-frequency amplified response to lateral ground motion of the tank shell and roof together with a portion of the liquid contents that moves in unison with the shell, and (2) the convective component is the low-frequency amplified response of a portion of the liquid contents in the fundamental sloshing mode. The design requires the determination of the hydrodynamic mass associated with each mode and the lateral force and overturning moment applied to the shell resulting from the response of the masses to the design acceleration.

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The natural period of the structure T_i is very small and is assumed to be zero for the general procedure. For the site-specific procedure, T_i shall be the natural period of the shell-fluid system. The first mode sloshing wave period T_c shall be determined by the equation

$$T_c = 2\pi \sqrt{\frac{D}{3.68g \tanh\left(\frac{3.68H}{D}\right)}} \qquad \text{(Eq 14-14)}$$

where:

T_c = first mode sloshing wave period in seconds
D = tank diameter in feet (m)
H = distance from bottom of shell to MOL in feet (m)

Other symbols have been previously defined in this section.

14.3.2 *Design overturning moment at the bottom of the shell.*

14.3.2.1 Design overturning moment at the bottom of the shell. The design overturning moment at the bottom of the shell caused by horizontal design acceleration is the SRSS combination of the impulsive and convective components and shall be determined by the equation

$$M_s = \sqrt{\left[A_i\left(W_s X_s + W_r H_t + W_i X_i\right)\right]^2 + \left[A_c W_c X_c\right]^2} \qquad \text{(Eq 14-15)}^*$$

where:

M_s = design overturning moment at the bottom of the shell due to horizontal design acceleration, in foot-pounds
A_i = impulsive design acceleration from Eq 14-12, stated as a multiple (decimal) of g
A_c = convective design acceleration from Eq 14-13, stated as a multiple (decimal) of g
W_s = total weight of tank shell and significant appurtenances in pounds
W_r = total weight of the tank roof, including framing and knuckle, plus permanent loads, if specified, in pounds
W_i = weight of effective mass of tank contents that moves in unison with the tank shell (effective impulsive weight) in pounds (Sec. 14.3.2.2)

* For equivalent metric equation, see Sec. 14.8.

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W_c = weight of effective mass of the first mode sloshing contents of the tank (effective convective weight) in pounds (Sec. 14.3.2.2)

X_s = height from the bottom of the shell to center of gravity of the shell in feet

H_t = total height of the shell in feet

X_i = height from the bottom of the shell to the centroid of lateral seismic force applied to the effective impulsive weight W_i in feet (Sec. 14.3.2.2.2)

X_c = height from the bottom of the shell to the centroid of lateral seismic force applied to the effective convective weight W_c in feet (Sec. 14.3.2.2.2)

14.3.2.2 Effective weight of tank contents.

14.3.2.2.1 Effective impulsive and convective weights W_i and W_c, respectively, shall be determined by the following equations:

For $D/H \geq 1.333$:

$$W_i = \frac{\tanh\left(0.866\frac{D}{H}\right)}{0.866\frac{D}{H}}W_T \qquad \text{(Eq 14-16)}$$

For $D/H < 1.333$:

$$W_i = \left[1.0 - 0.218\frac{D}{H}\right]W_T \qquad \text{(Eq 14-17)}$$

For all proportions of D/H:

$$W_c = 0.230\frac{D}{H}\tanh\left(\frac{3.67H}{D}\right)W_T \qquad \text{(Eq 14-18)}$$

where:

W_T = total weight of tank contents in pounds (kg) determined by the equation

$$W_T = 62.4GH\left(\frac{\pi D^2}{4}\right) = 49GHD^2 \qquad \text{(Eq 14-19)}^*$$

* For equivalent metric equation, see Sec. 14.8.

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where:

G = specific gravity = 1.0 for water

The other symbols have been previously defined in this section.

14.3.2.2.2 Heights X_i and X_c from the bottom of the shell to the centroids of the lateral seismic forces applied to effective weights W_i and W_c, respectively, shall be determined by the following equations:

For $D/H \geq 1.333$:

$$X_i = 0.375H \qquad \text{(Eq 14-20)}$$

For $D/H < 1.333$:

$$X_i = \left[0.5 - 0.094\frac{D}{H}\right]H \qquad \text{(Eq 14-21)}$$

For all proportions of D/H:

$$X_c = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \qquad \text{(Eq 14-22)}$$

The symbols have been previously defined in this section.

14.3.2.2.3 Effective weights W_i and W_c and heights X_i and X_c may be determined by other analysis procedures that take into account the dynamic characteristics of the tank and contents.

14.3.3 *Design shear and overturning moment at the top of the foundation.*

14.3.3.1 Design shear at the top of the foundation. The design shear at the top of the foundation due to horizontal design acceleration is the SRSS combination of the impulsive and convective components and shall be determined by the equation

$$V_f = \sqrt{\left[A_i\left(W_s + W_r + W_f + W_i\right)\right]^2 + \left[A_c W_c\right]^2} \qquad \text{(Eq 14-23)}^*$$

* For equivalent metric equation, see Sec. 14.8

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where:

V_f = design shear at the top of the foundation due to horizontal design acceleration in pounds

W_f = total weight of tank bottom in pounds

The other symbols have been previously defined in this section.

14.3.3.2 Design overturning moment at the top of the foundation.

14.3.3.2.1 Tanks supported by ringwall or berm foundations. The design overturning moment at the top of the foundation for tanks supported by ringwall or berm foundations is equal to the moment at the bottom of the shell due to horizontal design accelerations M_s determined by Eq 14-15.

14.3.3.2.2 Tanks supported by mat or pile cap foundations. The design overturning moment at the top of the foundation for tanks supported by mat or pile cap foundations shall include the effects of varying bottom pressures and shall be determined by the equation

$$M_{mf} = \sqrt{\left[A_i\left(W_s X_s + W_r H_t + W_i X_{imf}\right)\right]^2 + \left[A_c W_c X_{cmf}\right]^2} \qquad \text{(Eq 14-24)}^*$$

For $D/H \geq 1.333$:

$$X_{imf} = 0.375\left[1.0 + 1.333\left(\frac{0.866\frac{D}{H}}{\tanh\left(0.866\frac{D}{H}\right)} - 1.0\right)\right]H \qquad \text{(Eq 14-25)}$$

For $D/H < 1.333$:

$$X_{imf} = \left[0.50 + 0.06\frac{D}{H}\right]H \qquad \text{(Eq 14-26)}$$

* For equivalent metric equation, see Sec. 14.8

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For all proportions of *D*/*H*:

$$X_{cmf} = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1.937}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \qquad \text{(Eq 14-27)}$$

where:

M_{mf} = design overturning moment across the entire base cross section due to horizontal design acceleration in foot-pounds

X_{imf} = height from the bottom of the shell to the centroid of lateral seismic force applied to the effective impulsive weight W_i adjusted to include the effects of varying bottom pressures in feet

X_{cmf} = height from the bottom of the shell to the centroid of lateral seismic force applied to the effective convective weight W_c adjusted to include the effects of varying bottom pressures in feet

The other symbols have been previously defined in this section.

14.3.4 *Seismic design requirements.*

14.3.4.1 Resistance to overturning. Resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell, weight of roof reaction on the shell *w*rs, and by the weight of a portion of the tank contents adjacent to the shell for self-anchored tanks, or by mechanical anchoring.

The resisting force is adequate for tank stability and the tank may be self-anchored provided the following requirements are met:

1. The overturning ratio *J* determined by Eq 14-28 is less than 1.54. The maximum width of annulus for determining the resisting force is 3.5 percent of the tank diameter *D*.

2. The shell compression satisfies Sec. 14.3.4.2.

3. The required thickness of the bottom annulus t_b does not exceed the thickness of the bottom shell ring per Sec. 14.3.4.1.2.

4. Piping flexibility requirements of Sec. 14.4 are satisfied. The overturning ratio *J* is given by the equation

$$J = \frac{M_s}{D^2\left[w_t\left(1 - 0.4A_v\right) + w_L\right]} \qquad \text{(Eq 14-28)}^*$$

* For equivalent metric equation, see Sec. 14.8.

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where:

J = overturning ratio

w_t = weight of the tank shell and portion of the roof reacting on the shell determined by Eq 14-33 in pounds per foot of shell circumference

w_L = maximum resisting weight of tank contents in pounds per foot of shell circumference that may be used to resist the shell overturning moment (Sec. 14.3.4.1.1)

A_v = vertical design acceleration (Sec. 14.3.4.3), stated as a multiple (decimal) of g

The other symbols have been previously defined in this section.

For $J < 0.785$, there is no shell uplift due to the overturning moment and the tank is self-anchored.

For $0.785 \leq J \leq 1.54$, there is shell uplift, but the tank is stable provided the shell compression requirements of Sec. 14.3.4.2 are satisfied.

For $J > 1.54$, the tank is not stable. Modify the bottom annulus, within the limits of t_b and L, or provide mechanical anchoring.

14.3.4.1.1 Maximum resisting force of the bottom annulus. For self-anchored tanks, the portion of the contents used to resist overturning is dependent on the assumed width of the bottom annulus. The bottom annulus may be the tank bottom or an attached annular stiffener plate. For self-anchored tanks, the resisting force of the bottom annulus shall be determined by the equation

$$w_L = 7.9 t_b \sqrt{F_y HG} \leq 1.28 HDG \qquad \text{(Eq 14-29)}^*$$

where:

t_b = design thickness of the bottom annulus, in inches

F_y = minimum specified yield strength of the bottom annulus, in pounds per square inch

G = as defined in Section 5

The other symbols have been previously defined in this section.

The equation for w_L applies whether or not a thickened bottom annulus is used.

14.3.4.1.2 Width of the bottom annulus. The bottom annulus may be thicker than the bottom shell course, but the thickness t_b used to calculate seismic

* For equivalent metric equation, see Sec. 14.8.

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stability shall not exceed the thickness of the bottom shell course. When a bottom annulus is required, the width of the bottom annulus shall be equal to or greater than the width determined by the equation

$$L = 0.216 t_b \sqrt{\frac{F_y}{HG}} \text{ in feet} \leq 0.035D \qquad \text{(Eq 14-30)}^*$$

where:

L = required width of the bottom annulus measured from the inside of the shell, in feet

The other symbols have been previously defined in this section.

If the required width of the bottom annulus L exceeds $0.035D$, the tank must be anchored.

14.3.4.2 Shell stresses.

14.3.4.2.1 Longitudinal shell compression for self-anchored tanks. The maximum longitudinal shell compression stress at the bottom of the shell when there is no uplift ($J < 0.785$) shall be determined by the equation

$$\sigma_c = \left[w_t \left(1 + 0.4 A_v\right) + \frac{1.273 M_s}{D^2} \right] \frac{1}{12 t_s} \qquad \text{(Eq 14-31)}^*$$

The maximum longitudinal shell compression stress at the bottom of the shell when there is uplift ($0.785 \leq J \leq 1.54$) shall be determined by the equation

$$\sigma_c = \left[\frac{w_t \left(1 + 0.4 A_v\right) + w_L}{0.607 - 0.18667 J^{2.3}} - w_L \right] \frac{1}{12 t_s} \qquad \text{(Eq 14-32)}^*$$

In Eq 14-31 and Eq 14-32,

σ_c = maximum longitudinal shell compression stress, in pounds per square inch

t_s = actual thickness of the bottom shell course less the specified corrosion allowance, if any, in inches

w_t = weight of the tank shell and portion of the roof reacting on the shell, in pounds per foot of shell circumference, determined by the equation

* For equivalent metric equation, see Sec. 14.8.

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$$w_t = \frac{W_s}{\pi D} + w_{rs} \qquad \text{(Eq 14-33)}^*$$

where:

w_{rs} = roof load acting on the shell, in pounds per foot of shell circumference.

Only permanent roof loads shall be included. Roof live load shall not be included.

The other symbols have been previously defined in this section.

The maximum longitudinal shell compression stress σ_c must be less than the seismic allowable stress σ_e, which is determined in accordance with Sec. 14.3.4.2.4.

14.3.4.2.2 Longitudinal shell compression for mechanically anchored tanks. When mechanically anchored, the maximum longitudinal compression stress at the bottom of the shell shall be determined by Eq 14-31.†

14.3.4.2.3 Hoop shell tension. Hydrodynamic seismic hoop tensile stresses shall be determined by the following equations:

$$\sigma_s = \frac{\sqrt{N_i^2 + N_c^2 + (N_h A_v)^2}}{t_s} \qquad \text{(Eq 14-34)}^*$$

For $D/H \geq 1.333$:

$$N_i = 4.5 A_i GDH \left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right] \tanh\left(0.866 \frac{D}{H}\right) \qquad \text{(Eq 14-35)}^*$$

* For equivalent metric equation, see Sec. 14.8.

† The application of Eq 14-31 to anchorage analysis is a simplified design procedure. Depending on the site location and importance of the structure under consideration, the designer may wish to consider using a more rigorous anchorage analysis procedure.

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For $D/H < 1.333$ and $Y < 0.75D$:

$$N_i = 2.77 A_i G D^2 \left[\frac{Y}{0.75D} - 0.5 \left(\frac{Y}{0.75D} \right)^2 \right] \quad \text{(Eq 14-36)}^*$$

For $D/H < 1.333$ and $Y \geq 0.75D$:

$$N_i = 1.39 A_i G D^2 \quad \text{(Eq 14-37)}^*$$

For all proportions of D/H:

$$N_i = \frac{0.98 A_c G D^2 \cosh\left[\frac{3.68(H - Y)}{D} \right]}{\cosh\left[\frac{3.68H}{D} \right]} \quad \text{(Eq 14-38)}^*$$

where:

σ_s = hydrodynamic hoop tensile stress, in pounds per square inch
N_i = impulsive hoop tensile force, in pounds per inch
N_c = convective hoop tensile force, in pounds per inch
N_h = hydrostatic hoop tensile force
= $2.6GYD$ pounds per inch
= ($4{,}895GYD$ N/m)
t_s = actual thickness of the shell ring under consideration less the specified corrosion allowance, if any, in inches
Y = distance from MOL to the point under consideration, in feet (m) (positive down)

The other symbols have been previously defined in this section.

The hydrodynamic hoop tensile stresses σ_s shall be added to the hydrostatic stress in determining the total hoop tensile stress.

14.3.4.2.4 Allowable shell stresses. Allowable shell plate stresses in tension for the material of construction shall be based on the allowable stress in Section 5 as applicable. A one-third increase in basic allowable stress is permitted for seismic loading in accordance with Sec. 5.3.1.

* For equivalent metric equation, see Sec. 14.8.

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In compression, the effect of internal hydrostatic pressure on increasing buckling allowable stresses shall be included with a safety factor of 2.0 in the design of self-anchored tanks subjected to seismic loading. The seismic allowable longitudinal shell compression stress shall be determined by the following equations:

For self-anchored tanks:

$$\sigma_e = 1.333\left(\sigma_a + \frac{\Delta\sigma_{cr}}{2}\right) \qquad \text{(Eq 14-39)}$$

For mechanically anchored tanks:

$$\sigma_e = 1.333\sigma_a \qquad \text{(Eq 14-40)}$$

where:

σ_e = seismic allowable longitudinal shell compression stress, in pounds per square inch

σ_a = allowable compression stress f_s from Sec. 5.4.2, in pounds per square inch

$\Delta\sigma_{cr}$ = critical buckling stress increase for self-anchored tanks due to pressure, in pounds per square inch, determined by the equation

$$\Delta\sigma_{cr} = \frac{\Delta C_c E t}{R} \qquad \text{(Eq 14-41)}^*$$

ΔC_c = pressure-stabilizing buckling coefficient in accordance with the following equations:

For $\frac{P}{E}\left(\frac{R}{t}\right)^2 \leq 0.064$:

$$\Delta C_c = 0.72\left[\frac{P}{E}\left(\frac{R}{t}\right)^2\right]^{0.84} \qquad \text{(Eq 14-42)}^*$$

* For equivalent metric equation, see Sec. 14.8.

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For $\frac{P}{E}\left(\frac{R}{t}\right)^2 > 0.064$:

$$\Delta C_c = 0.045 \ln\left[\frac{P}{E}\left(\frac{R}{t}\right)^2 + 0.0018\right] + 0.194 \leq 0.22 \qquad \text{(Eq 14-43)}^*$$

E = modulus of elasticity, 29,000,000 in pounds per square inch
t = actual thickness of the plate under consideration less the specified corrosion allowance, if any, in inches
P = hydrostatic pressure at the point of consideration, in pounds per square inch
R = radius of the tank, in inches
ln = the natural logarithm function

14.3.4.3 Vertical design acceleration. The design of the tank and anchorage shall include load effects from vertical design acceleration A_v equal to $0.48S_{DS}$ for responses limited by buckling (Eq 14-31 and Eq 14-32) and $0.19S_{DS}$ for response not limited by buckling (Eq 14-28 and Eq 14-44) except as permitted in Sec. 14.3.4.5. Load effects from vertical design acceleration shall be combined with load effects from horizontal design acceleration by the direct sum method with load effects from vertical design acceleration being multiplied by 0.40, or by the SRSS method, unless otherwise specified.

14.3.4.4 Roof framing and columns. Seismic considerations shall be included in the design of roof framing and columns when specified. The live load used for horizontal and vertical seismic design of roof framing and columns shall be specified. When live load is specified for seismic design, it shall also be used to reduce uplift due to overturning (Eq 14-33). When seismic design of roof framing and columns is required, the design of columns shall include acceleration and lateral water loads. Seismic beam-column design shall be based upon the allowable stresses set forth in ANSI/AISC 360-10, increased one-third for seismic loading.

14.3.4.5 Sliding check. When a sliding check is specified the allowable lateral shear shall be determined by the equation

$$V_{ALLOW} = 0.4\left[W_s + W_r + W_i + W_c\right](1 - 0.4A_v) \qquad \text{(Eq 14-44)}^*$$

* For equivalent metric equation, see Sec. 14.8.

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where:

V_{ALLOW} = allowable lateral shear, in pounds

The other symbols have been previously defined in this section.

The allowable lateral shear shall be equal to or greater than the design shear at the top of the foundation due to horizontal design acceleration V_f determined by Eq 14-23, or additional shear resistance with a capacity of at least V_{NET} (Sec. 5.9.6.1) must be provided.

Sec. 14.4 Freeboard

Sloshing shall be considered in determining the freeboard above the MOL. Freeboard is defined as the distance from the MOL to the lowest level of the roof framing. The sloshing wave height shall be determined by the equation

$$d = 0.42DA_f \qquad \text{(Eq 14-45)}$$

where:

d = sloshing wave height above MOL, in feet

A_f = convective design acceleration for sloshing, stated as multiple (decimal) of g. The convective design acceleration for sloshing shall be determined by equations Eq 14-46 through Eq 14-49, as applicable.

The design freeboard shall be determined by Eq 14-45 and the requirements of Table 6. If the freeboard provided is less than the design freeboard, the effective water masses W_i and W_c shall be adjusted to include the confined portion of the sloshing mass as additional impulsive mass and the roof and supporting members shall be designed to contain the sloshing liquid and to resist the equivalent hydrostatic pressure for a water column equal to the design freeboard less the actual freeboard.

Table 6 Minimum freeboard requirements*

	Risk Category		
S_{DS}	I & II	III	IV
$S_{DS} < 0.33g$	None	None	d^*
$S_{DS} \geq 0.33g$	None	$0.7d^*$	d^*

* If the freeboard provided is less than the design freeboard, the design of the tank roof, and supporting structure all have additional requirements specified in Sec. 14.4.

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For Risk Categories I, II, and III:

When $T_c \leq 4$: $$A_f = \frac{KS_{D1}I_E}{T_c} \quad \text{(Eq 14-46)}$$

When $T_c > 4$: $$A_f = \frac{4KS_{D1}I_E}{T_c^2} \quad \text{(Eq 14-47)}$$

For Risk Category IV:

When $T_c \leq T_L$: $$A_f = \frac{KS_{D1}}{T_c} \quad \text{(Eq 14-48)}$$

When $T_c > T_L$: $$A_f = \frac{KS_{D1}T_L}{T_c^2} \quad \text{(Eq 14-49)}$$

K = damping scaling factor
= 1.5 to convert spectrum from 5 percent damping to 0.5 percent damping
S_{D1} = design earthquake spectral response acceleration, 5 percent damped, at 1-s period, stated as a multiple (decimal) of g
I_E = seismic importance factor from Table 2
T_c = first mode sloshing wave period, in seconds
T_L = region-dependent transition period for longer period ground motion, in seconds, from ASCE 7, Figures 22-12 through 22-16

Sec. 14.5 Piping Connections

14.5.1 *Flexibility.* Design of the piping system connected to the tank shall consider the potential movement of the connection points during earthquakes and provide sufficient flexibility to avoid release of the tank contents due to failure of the piping system. The piping system and supports shall be designed so as not to impart significant mechanical loading on the attachment to the tank. Mechanical devices that add flexibility such as bellows, expansion joints, and other flexible apparatus may be used when they are designed for the seismic displacements and defined operating pressure.

Unless otherwise specified, piping systems shall provide for the minimum design displacements in Table 7 at working stress levels (with one-third allowable

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Table 7 Minimum design displacements for piping attachments

Condition	Displacement *in.*	(*mm*)
Mechanically-anchored tanks		
Upward vertical displacement relative to support or foundation	1	(25)
Downward vertical displacement relative to support or foundation	0.5	(13)
Horizontal displacement (radial and tangential) relative to support or foundation	0.5	(13)
Self-anchored tanks		
Upward vertical displacement relative to support or foundation when $J \leq 0.785$	1	(25)
Upward vertical displacement relative to support or foundation when $J > 0.785$	4	(102)
Downward vertical displacement relative to support or foundation when the tank is supported by a ringwall or mat foundation	0.5	(13)
Downward vertical displacement relative to support or foundation when the tank is supported by a berm foundation	1	(25)
Horizontal displacement (radial and tangential) relative to support or foundation	2	(51)

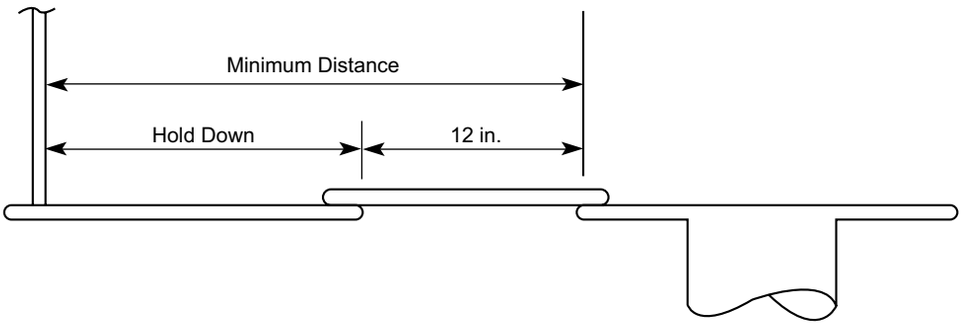


Figure 4 Bottom piping connection of a self-anchored tank with steel bottom

stress increase for seismic loads) in the piping, supports, and tank connection. The values given in Table 7 do not include the influence of relative movements of the foundation and piping anchorage points caused by foundation movements such as settlement. The effects of foundation movements shall be included in the design of the piping system. When $S_{DS} \leq 0.1$, the values in Table 7 may be reduced to 70 percent of the values shown.

14.5.2 *Bottom connection for self-anchored tanks.* The bottom connection for a self-anchored tank, if provided, shall be located inside the shell a sufficient distance to minimize damage by uplift. As a minimum, the distance measured to the edge of the connection reinforcement shall be the required width of the bottom annulus determined by Eq 14-34 plus 12 in. (305 mm) as shown in Figure 4.

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Sec. 14.6 Foundation Considerations

14.6.1 *Mechanically anchored tanks.* Foundations for mechanically anchored tanks shall be designed for maximum uplift and overturning bearing pressure. Water load above the foundation may be used for uplift resistance of the foundation, provided the foundation is designed for applicable eccentric loads. For calculating anchorage of the tank, water load resistance shall not be used.

14.6.2 *Self-anchored tanks.* Design shell compression loads on the foundation should be determined using the same method as for the mechanically anchored tank condition (Eq 14-35). This assumption does not permit larger response modification factors R_i and R_c than permitted for self-anchored tanks.

14.6.3 *Tank vaults.* If a vault or ringwall penetration exists, that portion under the shell shall be designed to carry the peak calculated shell load on its unsupported spans determined from Sec. 14.3.4.2.1 or Sec.14.3.4.2.2 as appropriate.

Sec. 14.7 Tensile Straps

Horizontal joints on tanks with formed flange construction could require tensile straps to transfer vertical tension loads. The design procedure in Sec. 15.5 shall be utilized to determine if tensile straps are required for these joints to resist forces generated under load combinations including seismic loads specified by this section.

Sec. 14.8 Equivalent Metric Equations

Metric equivalents of equations presented in Section 14 are as follows:

Equation Number	Equivalent Metric Equation	Variable	Metric Units
14-15	$M_s = 9.81\sqrt{\left[A_i\left(W_sX_s + W_rH_t + W_iX_i\right)\right]^2 + \left[A_cW_cX_c\right]^2}$	M_s W_s, W_r W_i, W_c X_s, H_t X_i, X_c	N-m kg kg m m
14-19	$W_T = 785.4\,GHD^2$	W_T D, H	kg m
14-23	$V_f = 9.81\sqrt{\left[A_i\left(W_s + W_r + W_f + W_i\right)\right]^2 + \left[A_cW_c\right]^2}$	V_f W_s, W_r W_f, W_i W_c	N kg kg kg
14-24	$M_{mf} = 9.81\sqrt{\left[A_i\left(W_sX_s + W_rH_t + W_iX_{imf}\right)\right]^2 + \left[A_cW_cX_{cmf}\right]^2}$	M_{mf} W_s, W_r W_i, W_c X_s, H_t X_{imf}, X_{cmf}	N-m kg kg m m

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Equation Number	Equivalent Metric Equation	Variable	Metric Units
14-28	$J = \dfrac{M_s}{D^2\left[w_t\left(1 - 0.4A_v\right) + w_L\right]}$	M_s D w_t, w_L	N-m m N/m
14-29	$w_L = 99t_b\sqrt{F_y HG} \le 201.1HDG$	w_L t_b F_y H,D	N/m mm MPa m
14-30	$L = 0.0172t_b\sqrt{\dfrac{F_y}{HG}} \le 0.035D$	H,L,D t_b F_y	m mm MPa
14-31	$\sigma_c = \left[w_t\left(1 + 0.4A_v\right) + \dfrac{1.273M_s}{D^2}\right]\left[\dfrac{1}{1{,}000t_s}\right]$	σ_c w_t M_s D t_s	MPa N/m N-m m mm
14-32	$\sigma_c = \left[\dfrac{w_t\left(1 + 0.4A_v\right) + w_L}{0.607 - 0.18667J^{2.3}} - w_L\right]\left[\dfrac{1}{1{,}000t_s}\right]$	σ_c w_t, w_L t_s	MPa N/m mm
14-33	$w_t = \dfrac{9.81W_s}{\pi D} + w_{rs}$	w_t, w_{rs} W_s D	N/m kg m
14-34	$\sigma_s = \dfrac{\sqrt{N_i^2 + N_c^2 + \left(N_h A_v\right)^2}}{1{,}000t_s}$	σ_s N_i, N_c, N_h t_s	MPa N/m mm
14-35	$N_i = 8{,}480A_i GDH\left[\dfrac{Y}{H} - 0.5\left(\dfrac{Y}{H}\right)^2\right]\tanh\left(0.866\dfrac{D}{H}\right)$	N_i D,H,Y	N/m m
14-36	$N_i = 5{,}220A_i GD^2\left[\dfrac{Y}{0.75D} - 0.5\left(\dfrac{Y}{0.75D}\right)^2\right]$	N_i D,Y	N/m m
14-37	$N_i = 2{,}620A_i GD^2$	N_i D	N/m m
14-38	$N_c = \dfrac{1{,}850A_c GD^2\cosh\left[\dfrac{3.68\left(H - Y\right)}{D}\right]}{\cosh\left(\dfrac{3.68H}{D}\right)}$	N_c D,H,Y	N/m m
14-41	$\Delta\sigma_{cr} = \dfrac{\Delta C_c Et}{R}$	$\Delta\sigma_{cr}, E$ t,R	MPa mm

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Equation Number	Equivalent Metric Equation	Variable	Metric Units
14-42	For $\frac{P}{E}\left(\frac{R}{t}\right)^2 \le 0.064 : \Delta C_c = 0.72\left[\frac{P}{E}\left(\frac{R}{t}\right)^2\right]^{0.84}$	P,E t,R	MPa mm
14-43	For $\frac{P}{E}\left(\frac{R}{t}\right)^2 > 0.064$ $\Delta C_c = 0.045 \ln\left[\frac{P}{E}\left(\frac{R}{t}\right)^2 + 0.0018\right] + 0.194 \le 0.22$	P,E t,R	MPa mm
14-44	$V_{ALLOW} = 9.81 \tan 30^\circ \left[W_s + W_r + W_i + W_c\right](1 - 0.4A_v)$	V_{ALLOW} W_s,W_r W_i,W_c	N kg kg

SECTION 15: WIND DESIGN

Sec. 15.1 Wind Pressure

15.1.1 *Wind load.* Wind load shall be in accordance with the ASCE 7 directional procedure for other structures and building appurtenances, except as modified herein. Wind loads are reduced to service-level loads by the use of strength-level to service level factor λ_w.

15.1.1.1 Basic wind speed. The basic wind speed V for areas outside hurricane-prone regions and special wind regions shall be in accordance with ASCE 7, Figure 26.5, and the assigned risk category.

15.1.1.1.1 Special wind regions. The basic wind speed for mountainous terrain, gorges, and special wind regions shown in ASCE 7, Figure 26.5, shall be specified. The basic wind speed shall be adjusted to account for higher local wind speeds. When higher local wind speeds are applicable, the basic wind speed shall be adjusted, and the wind speed adjustment shall be specified. The adjustment shall be based on meteorological information and an estimate of the basic wind speed in accordance with ASCE 7.

15.1.1.1.2 Hurricane-prone regions. Hurricane-prone regions are defined as Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed for Risk Category II structures is greater than 115 mph (185 km/hr), Hawaii, Puerto Rico, Guam, Virgin Islands, and American Samoa. The basic wind speed for hurricane-

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prone regions shall be in accordance with ASCE 7, Figure 26.5, or it may be determined from simulation techniques in accordance with ASCE 7.

15.1.1.2 Exposure category. The exposure category shall be based on the most severe upwind exposure. The upwind exposure shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities. For sites located in the transition zone between exposure categories, the category resulting in the highest wind load shall be used. Exposure C shall be used unless otherwise specified.

15.1.1.2.1 Surface roughness categories. Surface Roughness C is characterized by open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m). Surface Roughness C includes flat open country and grasslands. Surface Roughness D is characterized by flat, unobstructed areas and water surfaces. Surface Roughness D includes smooth mud flats, salt flats, and unbroken ice.

15.1.1.2.2 Exposure categories. Exposure C shall apply for all cases where Exposure D does not apply. Exposure D shall apply where Surface Roughness D prevails in the upwind direction for a distance greater than 5,000 ft (1,524 m) or 20 times the total structure height, whichever is greater. Exposure D shall also apply where Surface Roughness C prevails immediately upwind of the site, and the site is within a distance of 600 ft (183 m) or 20 times the total structure height, whichever is greater, from an Exposure D condition.

15.1.1.3 Topographic effects. The topographic factor K_{zt} accounts for increased wind speed due to abrupt changes in the general topography. Determination of the topographic factor shall comply with ASCE 7. The topographic factor shall be 1.0 unless otherwise specified.

15.1.1.4 Gust effects. The gust-effect factor G accounts for loading effects in the along-wind direction due to wind turbulence-structure interaction. The gust-effect factor for rigid structures (i.e., structures with a fundamental frequency equal to or greater than 1 Hz) shall be taken as 0.85. The gust-effect factor for flexible structures (i.e., structures with a fundamental frequency less than 1 Hz) shall be calculated in accordance with ASCE 7 and shall not be less than 0.85.

15.1.1.5 Velocity pressure. Velocity pressure q_z at height z shall be determined using the equation

$$q_z = \lambda_w \left[0.00256 K_z K_{zt} K_d V^2 \right] \qquad \text{(Eq 15-1)}^*$$

* For equivalent metric equation, see Sec. 15.6.

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where:

q_z = velocity pressure evaluated at height, z, of the centroid of the projected area of each portion of the structure contained within each height zone defined in Table 8, in pounds per square foot.

λ_w = strength-level to service level factor for wind load

= 0.6

K_z = velocity pressure exposure coefficient evaluated at height z of the centroid of the projected area (see Table 8).

K_{zt} = topographic factor in accordance with section 15.1.1.3

K_d = directionality factor

= 1.0

V = basic wind speed, in miles per hour (m/s) (ASCE 7 Figure 26.5)

z = height above finished grade, in feet (m)

15.1.1.6 Design wind pressure. Design wind pressure, p_w, shall be determined using the equation

Table 8 Velocity pressure exposure coefficient Kz*

Height Above Ground Level z		Velocity Pressure Exposure Coefficient K_z*	
(ft)	(m)	Exposure C	Exposure D
0–15	0–4.6	0.85	1.03
20	6.1	0.90	1.08
25	7.6	0.94	1.12
30	9.1	0.98	1.16
40	12.2	1.04	1.22
50	15.2	1.09	1.27
60	18.0	1.13	1.31
70	21.3	1.17	1.34
80	24.4	1.21	1.38
90	27.4	1.24	1.40
100	30.5	1.26	1.43
120	36.6	1.31	1.48
140	42.7	1.36	1.42
160	48.8	1.39	1.55
180	54.9	1.43	1.58
200	61.0	1.46	1.58
250	76.2	1.53	1.68
300	91.4	1.59	1.73

* Velocity pressure exposure coefficient K_z may be calculated in accordance with ASCE 7, Table 29.3-1.

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$$p_w = q_z G\ C_f \geq 30 C_f \qquad \text{(Eq 15-2)}^*$$

where:

p_w = wind pressure applied to projected area on a vertical plane in pounds per square foot

G = gust effect factor in accordance with Sec. 15.1.1.4

C_f = force coefficient (see Table 9) in accordance with section 15.1.1.6.1

15.1.1.6.1 Wind force coefficient C_f. Surfaces exposed to wind shall be classified according to shape as cylindrical, conical, double-curved, flat, or a structural shape. The quantity $d\sqrt{q_z}$ shall be based on the average velocity pressure applied to the surface.

Cylindrical surfaces without protruding longitudinal elements shall be classified moderately smooth. Cylindrical surfaces with protruding longitudinal elements (e.g., pilasters and fluted support structure) shall be classified according to the ratio d'/d.

Wind force coefficient C_f shall be in accordance with Table 9. Wind force coefficients for shapes and conditions not addressed in Table 9 shall be in accordance with ASCE 7.

Table 9 Force coefficient C_f

		Wind Force Coefficient C_f		
Shape of Surface	Type of Surface	$l/d \leq 1$	$l/d = 7$	$l/d \geq 25$
Cylindrical or conical with apex angle < 15 degrees ($d\sqrt{q_z} > 2.5$)	Moderately smooth	0.5	0.6	0.7
	Rough (d'/d =0.02)	0.7	0.8	0.9
	Very rough (d'/d =0.08)	0.8	1.0	1.2
Cylindrical or conical with apex angle < 15 degrees ($d\sqrt{q_z} \leq 2.5$)	All	0.7	0.8	1.2
Double curved or conical with apex angle ≥ 15 degrees	All		0.5	
Flat	All		1.3	
Structural shape	All		1.8	

d = diameter of cylindrical surface or average diameter of conical surface, in feet
d' = depth of protruding elements such as pilaster or fluted sections, in feet
l = length of cylindrical or conical surface in the plane that contains the axis of revolution, in feet

* For equivalent metric equation, see Sec. 15.6.

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15.1.1.7 Wind load *W*. Wind load *W* shall be determined by applying the design wind pressure p_w to the projected area A_f of the tank and support structure. Wind loads shall be applied at the centroids of the projected areas.

15.1.1.7.1 Wind-structure interaction. The effects of wind-structure interaction, such as vortex shedding, shall be considered for tall standpipes in accordance with ASCE 7 slender standpipes ($H/D > 5$) where the lateral deflection due to wind loads is greater than 0.02 times the tank diameter. For the purposes of this *H/D* limit, *H* is defined as the overall height of the standpipe.

Sec. 15.2 Anchor Check

When the calculated net uplift force calculated for wind forces results in a negative value, no uplift anchorage is required. See Eq 5-13.

Sec. 15.3 Sliding Check

When a sliding check for a self-anchored tank is specified, a coefficient of friction equal to 0.4 shall be used. The total wind shear force shall be less than the resisting friction force in accordance with the following formula:

$$p_{aw}HD \leq 0.4W_{tot} \qquad \text{(Eq 15-3)}^*$$

where:

p_{aw} = average wind pressure acting over *H*, in pounds per square foot. The average wind pressure shall be based on wind pressures defined in Sec. 15.1.

H = height of the cylindrical portion of the tank shell, in feet

D = diameter of the tank shell, in feet

W_{tot} = total weight of empty tank shell, roof, steel bottom, and permanent accessories, in pounds

Sec. 15.4 Top and Intermediate Shell Girders

15.4.1 *Top girder.* A tank without a roof shall have a top girder or angle having a minimum section modulus as determined by the formula:

$$S = 0.0001HD^2\left(p_{aw}/18\right) \qquad \text{(Eq 15-4)}^*$$

where:

S = the minimum required section modulus of the top angle or girder (including a portion of the tank shell for a distance of the lesser

* For equivalent metric equation, see Sec. 15.6.

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of $16t$ or $0.78(Rt)^{1/2}$ below and, if applicable, above the ring attachment to the shell) in cubic inches

t = the tank wall thickness at the girder attachment location in inches

R = the nominal radius of the tank shell in inches

H = the height of the cylindrical portion of the tank shell in feet

D = the tank diameter in feet

p_{aw} = the average wind pressure acting over H in pounds per square foot

The average wind pressure shall be based on wind pressures defined in Sec. 15.1. (Note: When p_{aw} is 18 pounds per square foot or less, the term (p_{aw}/18) is to be unity.)

15.4.1.1 The total vertical leg of the top girder or angle may be used in the computations, provided that the vertical leg width does not exceed the width-to-thickness ratios set forth in Sec. 5.3.

15.4.1.2 Where glass-fused-to-steel coated wind girders are used, the required minimum section modulus, S, in Eq 15-4 shall be multiplied by the greater of 30,000 psi/F'_y (206 MPa/F'_y) and 1.0.

15.4.2 *Intermediate girders.* The following formula shall be used to determine whether intermediate girders are required between the roof or top girder and the bottom:

$$h = \frac{10.625 \times 10^6 t}{p_{aw}\left(\frac{D}{t}\right)^{1.5}} \qquad \text{(Eq 15-5)}^*$$

where:

h = vertical distance between the intermediate wind girder and the top angle of the shell or the top wind girder of an open-top tank in feet

D = tank diameter in feet

t = average shell thickness for the vertical distance h in inches

15.4.2.1 In determining the maximum height of unstiffened shell, an initial calculation shall be made using the thickness of the top shell course. Additional calculations shall be based on the average thickness obtained by including part or all of the next lower course or courses, until the calculated h is equal to or less than the height of shell used in determining the average thickness. If h continues

* For equivalent metric equation, see Sec. 15.6.

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to calculate greater than the height of the shell used in determining the average thickness, no intermediate girder is required.

15.4.2.2 After establishing the location of the first intermediate girder, if required, repeat the procedure for additional intermediate girders, using the preceding intermediate girder as the top of the tank. Locating the intermediate wind girder at the maximum spacing calculated by the preceding rules will usually result in a shell below the intermediate wind girder having a greater stability against wind loading than the shell above the intermediate girder. The girder may be located at a spacing less than the maximum spacing, but the lower shell must be checked for adequacy against the maximum wind pressure, as previously described or in the following alternative subparagraphs.

1. Change the width W of each shell course into a transposed width W_{tr} of shell course, having a uniform thickness, by the following relationship:

$$W_{tr} = W\left(\frac{t_{uniform}}{t_{actual}}\right)^{2.5} \qquad \text{(Eq 15-6)}$$

where:

$t_{uniform}$ = uniform thickness into which the entire shell will be transformed in inches (mm)

t_{actual} = actual thickness of the plate course being transformed in inches (mm)

2. The sum of the transposed width of each course will give the height of an equivalent transformed shell. For equal stability above or below the intermediate wind girder, the girder should be located at the mid-height of the transformed shell. The location of the girder on the transformed shell shall be transposed to the actual shell by the foregoing thickness relationship, using the actual thickness of the shell course on which the girder will finally be located and all actual thicknesses above this course.

15.4.2.3 When intermediate girders are required, the minimum required section modulus shall be calculated using Eq 15-4.

Sec. 15.5 Tensile Straps

Horizontal joints on tanks with formed flange construction may require tensile straps to transfer vertical tension loads from wind and seismic forces.

The following formula shall be used to calculate the allowable tension load for horizontal flanged joints without a tensile strap:

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$$T = 9{,}600t^2\left(F_y / 36{,}000\right) \qquad \text{(Eq 15-7)}^*$$

where:

T = allowable tension load in pounds per inch

t = thinnest shell thickness utilized in the horizontal joint being analyzed in inches

F_y = minimum published yield strength utilized for steel in the horizontal joint being analyzed

Eq 15-7 is applicable for formed flange horizontal seams having ½-in. (13-mm) diameter bolts on 2-in. (51-mm) spacing at a bolt circle 1 in. (25 mm) larger than the outside radius of the shell. No increase in allowable load is permitted. When the actual tension load exceeds the allowable tension load, tensile straps are required, and shall be designed in accordance with Sec. 5.5.

Sec. 15.6 Equivalent Metric Equations

Metric equivalents of equations presented in Section 15 are as follows:

Equation Number	Equivalent Metric Equation	Variable	Metric Units
15-1	$q_z = \lambda_w\left[0.613K_zK_{zt}K_dV^2\right]$	q_z	N/m^2
		V	m/sec
15-2	$P_w = q_zGCf \geq 1{,}436Cf$	$P_w\, q_z$	N/m^2
15-3	$p_{aw}HD \leq 9.81W_{tot}\,(0.4)$	p_{aw}	N/m^2
		H,D	m
		W_{tot}	kg
15-4	$S = 0.06713HD^2\,p_{aw}$	S	mm^3
		H	m
		D	m
		p_{aw}	N/m^2
15-5	$h = \dfrac{8{,}025t}{p_{aw}\left(\dfrac{D}{t}\right)^{1.5}}$	h,	m
		p_{aw}	N/m^2
		D	m
		t	mm
15-7	$T = 2.61t^2\left(F_y/248\right)$	T	kN/m
		t	mm
		F_y	MPa

* For equivalent metric equation, see Sec. 15.6.

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APPENDIX A

Commentary for Factory-Coated Bolted Carbon Steel Tanks for Water Storage

This appendix is for information only and is not a part of ANSI/AWWA D103. Numbers following the "A" reference the applicable section in the body of the standard.

SECTION A.1: GENERAL (REFER TO SECTION 1 OF ANSI/ AWWA D103)

Sec. A.1.1 Items Not Described

This standard does not address disinfecting tanks. Refer to ANSI/ AWWA C652 for disinfection requirements.

SECTION A.2: REFERENCES (REFER TO SECTION 2 OF ANSI/ AWWA D103)

To the extent specified in this standard, the requirements contained in the references listed in Section 2 are part of the standard. The following references contain useful information related to steel tanks for water storage:

1. ACI 355.1R-91—Report on Anchorage to Concrete.

2. ASCE 7-10—Minimum Design Loads for Buildings and Other Structures.

3. AWWA Manual M42—*Steel Water-Storage Tanks.*

4. *Earthquake Engineering for Nuclear Reactors*. 1971. San Francisco, Calif.: J.A. Blume & Associates.

5. NFPA 22-03—Standard for Water Tanks for Private Fire Protection.

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SECTION A.4: MATERIALS (REFER TO SECTION 4 OF ANSI/AWWA D103)

Sec. A.4.2 Bolts, Nuts, and Anchor Bolts

A.4.2.2 Galvanizing. ASTM F3125, SAE J429 grade 8, and SAE J995 grade 8 high-strength fasteners shall not be hot-dip galvanized, due to possible hydrogen embrittlement. Unless certain procedures are utilized in the plating process, hydrogen embrittlement can occur and have negative effects on the expected strength capabilities of high-strength fasteners. Mechanical galvanizing has proven acceptable for high-strength fasteners, provided that the proper procedures are followed to prevent or eliminate hydrogen embrittlement.

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SECTION A.5: GENERAL DESIGN (REFER TO SECTION 5 OF ANSI/ AWWA D103)

Sec. A.5.3 Design Criteria

A.5.3.2 The modified steel properties, F'_y and F'_u, specified in Sec. 5.3.2.1 shall apply to the following sections and equations where applicable: Sec. 5.5.1, Eq 5-3, Sec. 5.5.3, Eq 5-4, Eq 5-5, Sec. 5.5.4, Sec. 5.7 (ANSI/AISC 360-10 designs), Eq 5-9, Sec. 5.9.5, Sec. 14.3.4.1.1, Eq 14-33, Sec. 14.3.4.1.2, Eq 14-34, Sec. 15.4.1, Eq 15-4, Sec. A.14.4.1, and Eq A14-5.

Sec. A.5.7 Roof Supports

When a soil investigation indicates that excessive differential settlement is expected, consideration should be given to the possibility that axial loads could develop in the rafters due to the differential settlements. Some possible solutions are:

1. Correct or improve the foundation conditions and specifications to eliminate the excessive differential settlements or to reduce the settlements to tolerable levels.
2. Provide a means to jack the columns and reset the structure elevation after the settlement has occurred.
3. Provide rafter details and connections that minimize axial loads due to the differential settlement. Rafters and connections shall be designed for the anticipated axial loads.

Erection stability of the roof structure system should be verified to ensure stability of the proposed erection sequence and methods.

Eq 5-12 is empirical and is set to allow the historical 60-in. (1,524-mm) spacing on a 0.094 in. (2.4-mm), grade 40 roof with a standard 25-psf (1.2-kPa) live load.

Sec. A.5.9 Foundation Anchor Bolts

Embedded anchor bolts of the "J" and "L" type are not recommended. The ACI document "Report on Anchorage to Concrete" (ACI 355.1R-91) states that bent, smooth, or deformed threaded bars have been known to straighten out in pullout tests.

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The allowable stresses for anchor bolts do not include allowance for prying action due to corner bearing on nuts. The placement tolerances of Sec. 13.6.3 are intended to minimize such prying action.

A.5.9.1.2.2 The pullout capacity of the anchor shall be reduced when the horizontal radius of adjacent pullout cones overlap and when the free edge distance is less than the horizontal radius of the pullout cone from the anchor.

A.5.9.6.1 Design loads. Only one-half of the anchors are considered effective for any direction of ground motion.

SECTION A.7: ACCESSORIES FOR TANKS (REFER TO SECTION 7 OF ANSI/AWWA D103)

Sec. A.7.2 Pipe Connections

It is impractical to design tank shells for excessively large piping loads. Piping should be designed to minimize the loads imposed on the shell.

Sec. A.7.3 Overflow

An overflow protects the tank from overpressure, overload, and possible catastrophic failure should the pumps or altitude 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.

Internal overflows, where a portion of the piping is within the tank container, are not recommended when tank usage and climatic conditions are such that ice damage may occur to the overflow pipe or its attachments.

Sec. A.7.4 Ladders

A.7.4.2 *Inside tank ladder.* Inside tank ladders are not recommended for cold climates where ice may form.

A.7.4.3 *Access to roof hatches and vents.* Regardless of the access protection provided to roof hatches and vents, weather conditions

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on tank roofs are extremely variable, and workers are expected to exercise good judgment and follow applicable regulations in matters of safety.

Sec. A.7.5 Safety Devices

Ladder cages, rest platforms, and guardrails are not recommended for use inside the tank. Cages may prevent access to the ladder rungs for escape from drowning in deep water. Platform and guardrail protrusions may be subjected to water turbulence and corrosion, harbor algae and contaminants, and disturb influent and effluent water flow.

Sec. A.7.7 Vent

Venting is recommended to minimize condensation on the underside of the roof.

An improperly vented tank may cause external pressures to act on the tank that can cause buckling even at a low-pressure differential.

Vent screens should be cleaned and pallets or relief mechanism should be checked for proper operation at least once a year but preferably each spring and fall.

Sec. A.7.8 Galvanic Corrosion

For tank components exposed to the stored water, construction with dissimilar metals more noble (stainless steel, copper, brass, and so on) than the carbon steel tank components to which they are attached is prohibited by this standard unless those dissimilar metal components are electrically isolated from the carbon steel tank at all points of connection. When dissimilar metal internal components are not electrically isolated and they are in contact with the water or condensation, corrosion of the carbon steel tank will occur. Experience has also shown that even non-isolated dissimilar metals above the MWL on open top tanks can cause dissimilar metals corrosion of carbon steel components. Also, non-isolated dissimilar metal components will interfere with the effectiveness of any cathodic protection systems that may be in place. In most cases, dissimilar metal components can be isolated from the carbon steel tank using alternate connection details. In cases where it is believed that they cannot be isolated from the tank, check with tank manufacturers and consultants for alternate

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details that can provide isolation. In cases where electrical isolation of dissimilar metal internal components below the overflow level has been determined to not be possible, then it is recommended that the dissimilar metal components be lined and coated with a suitable dielectric coating. It is the responsibility of the owner or specification writer to determine whether any dissimilar metals are specified and, if so, how the dissimilar metals are to be treated with respect to the coating system. For additional information on common interior lining systems for potable water tanks, refer to ANSI/AWWA D102—Coating Steel Water-Storage Tanks.

SECTION A.11: INSPECTION AND TESTING (REFER TO SECTION 11 OF ANSI/AWWA D103)

Sec. A.11.2 Testing

A.11.2.1 *Leak testing of the bottom.* Vacuum testing is performed by means of a suitable testing box. The open bottom is sealed against the tank surface by a sponge-rubber gasket. Suitable connections, valves, and gauges should be provided. The seam being tested is brushed with a soapsuds solution or linseed oil. In freezing weather, a nonfreezing solution may be necessary. The vacuum box is placed over the coated section of seam and a vacuum is applied to the box. The presence of a leak in the seam is indicated by bubbles or foam produced by air sucked through the seam. A vacuum can be drawn on the box by any convenient method, such as a connection to a gasoline or diesel engine intake manifold, or to an air ejector or special vacuum pump. The gauge should register a vacuum of at least 2 psi (13.8 kPa).

A.11.2.3 *Repair of leaks.* It is preferred that the proper repair of leaks be made while the water level is above the point being repaired.

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SECTION A.13: FOUNDATION DESIGN AND CONSTRUCTION (REFER TO SECTION 13 OF ANSI/ AWWA D103)

Sec. A.13.1 General Requirements

The proper design and installation of foundations for tanks are extremely important to ensure uniform and minimum settlement. Unequal settlement considerably changes distribution of stresses in the structure and may cause leakage or buckling of the tank components. The tops of foundations shall be located accurately at the proper elevation.

Sec. A.13.5 Foundation Design Details

A.13.5.1 *Height above ground.* A projection greater than 6 in. (152 mm) may be specified to facilitate site conditions (e.g., finish grade that slopes across the tank site). Serviceability issues (e.g., safe access to shell openings from grade) and design issues (e.g., increased overturning moment at the base of the foundation) related to projection must be considered.

SECTION A.14: SEISMIC DESIGN (REFER TO SECTION 14 OF ANSI/ AWWA D103)

Sec. A.14.1 General

In this standard, ASCE 7-10 is primarily used as the basis for the minimum seismic design loads.

A.14.1.1 *Scope.* In regions where the regulatory requirements for determining design ground motion differ from the ASCE 7 methods prescribed in the standard, the following methods may be utilized:

1. A response spectrum complying with the regulatory requirements may be used, provided it is based on, or adjusted to, a basis of 5 percent and 0.5 percent damping as required in the standard. A_i shall be based on the calculated impulsive period of the tank using

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the 5 percent damped spectrum. A_c shall be based on the calculated convective period using the 0.5 percent damped spectrum.

2. If no response spectra are prescribed and only the peak ground acceleration S_P is defined, the following substitutions shall apply:

$$S_S = 2.5S_P \quad \text{(Eq A14-1)}$$

$$S_1 = 1.25S_P \quad \text{(Eq A14-2)}$$

where:

S_S = mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-s period, stated as a multiple (decimal) of g

S_1 = mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-s period, stated as a multiple (decimal) of g

S_P = peak ground acceleration, 5 percent damped, stated as a multiple (decimal) of g

Sec. A.14.2 Design Earthquake Ground Motion

Ground motion accelerations, represented by response spectra and coefficients derived from these spectra, shall be determined in accordance with the general procedure of Sec. 14.2.7 or the site-specific procedure of Sec. 14.2.8. The general procedure in which spectral response acceleration parameters for the maximum considered earthquake ground motions are derived using Figures 5 through 18, modified by site coefficients to include local site effects, and scaled to design values, are permitted to be used for any structure except as specifically indicated in the standard. The site-specific procedure of Sec. 14.2.8 is permitted to be used for any structure and shall be used when specified or required by the standard.

A.14.2.1 *Risk Category.* Tanks are classified in the appropriate Risk Category based on the function and hazard to the public. A higher Risk Category may be specified to match the risk management approach for the tank or facility. Specifying a higher Risk Category increases the seismic importance factor I_E, used to define the design acceleration, and indirectly influences the performance level expected

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of the tank. Selection of the appropriate Risk Category should be by an individual who is familiar with the risk management goals of the facility and surrounding environment.

The governing regulatory requirements may differ from ASCE 7 and may use an Occupancy Category to define the importance of the structure.

Tanks serving the following types of applications may be assigned Risk Category IV, unless an alternative or redundant source is available:

1. Fire, rescue, and police stations.
2. Hospitals and emergency treatment facilities.
3. Power-generating stations or other utilities required as emergency backup facilities for Risk Category IV facilities.
4. Designated essential communication centers.
5. Water production, distribution, or treatment facilities required to maintain water pressure for fire suppression within the municipal or public domain (i.e., not industrial).

Tanks serving the following types of applications may be assigned Risk Category III, unless an alternative or redundant source is available:

1. Power-generating stations and other public utility facilities not included in Risk Category IV, but required for continued operation.
2. Water and wastewater treatment facilities required for primary treatment and disinfection for potable water.

Risk Category II is used for tanks not designated Risk Category IV or Risk Category III.

A.14.2.3 *Site Class.* The ground motions must be amplified when the founding soils are not rock. In previous editions, these adjustments only applied to the constant velocity and acceleration portions of the response. Since the mid-1990s, there have been dual site factors as found in ASCE 7 to define the influence of the soil on the shape and values of the ground motions. This section utilizes this ASCE 7 approach.

Tanks should not be located on Site Class E or F soils when there is a known potential for an active fault that can cause rupture of the ground surface at the tank.

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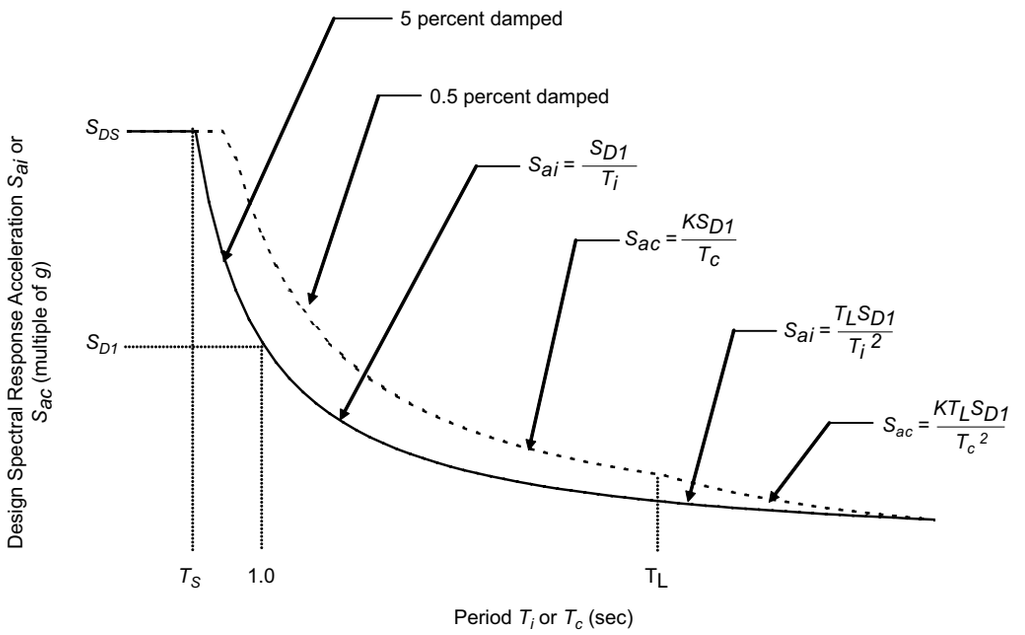


Figure A1 Design response spectra—general procedure

A.14.2.6.3 Design response spectra. The design response spectra for the general procedure (equations Eq 14-5 through Eq 14-9) are shown in Figure A1.

A.14.2.7 *Design response spectra—site-specific procedure.* The site-specific procedure is used to develop ground motions that are determined with higher confidence for the local seismic and site conditions than can be determined by using the general procedure of Sec. 14.2.6, and is required for tanks located on Site Class F soils.

A.14.2.7.6 Design response spectrum. 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-s period, except that it shall not be taken as less than 90 percent of the peak spectral acceleration at any period larger than 0.2 s. Similarly, the parameter S_{D1} shall be taken as the greater of the spectral acceleration at 1-s period or two times the spectral acceleration at 2-s period. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be taken as less than 80 percent of the values obtained from the general procedure of Sec. 14.2.6. The resulting site-specific design

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spectrum should be generated in accordance with Sec. 14.2.6.3.1 and should be smoothed to eliminate extreme humps and jagged variations.

A.14.2.7.6.1 Design response spectrum for impulsive components. For the site-specific procedure, the design spectral response acceleration for impulsive components S_{ai} may be limited to the acceleration that causes the tank to slide $S_{ai,slide}$. The impulsive acceleration that causes the tank to slide may be approximated by the equation

$$S_{ai,slide} = \tan 30^\circ \left(\frac{W_T}{W_i} \right) \qquad \text{(Eq A14-3)}$$

where:

$S_{ai,slide}$ = design spectral response acceleration that causes the tank to slide, 5 percent damped, stated as a multiple (decimal) of g

W_T = total weight of tank contents, in pounds, determined by Eq 14-19

W_i = weight of effective mass of tank contents that moves in unison with the tank shell, in pounds, determined by Eq 14-16 or Eq 14-17

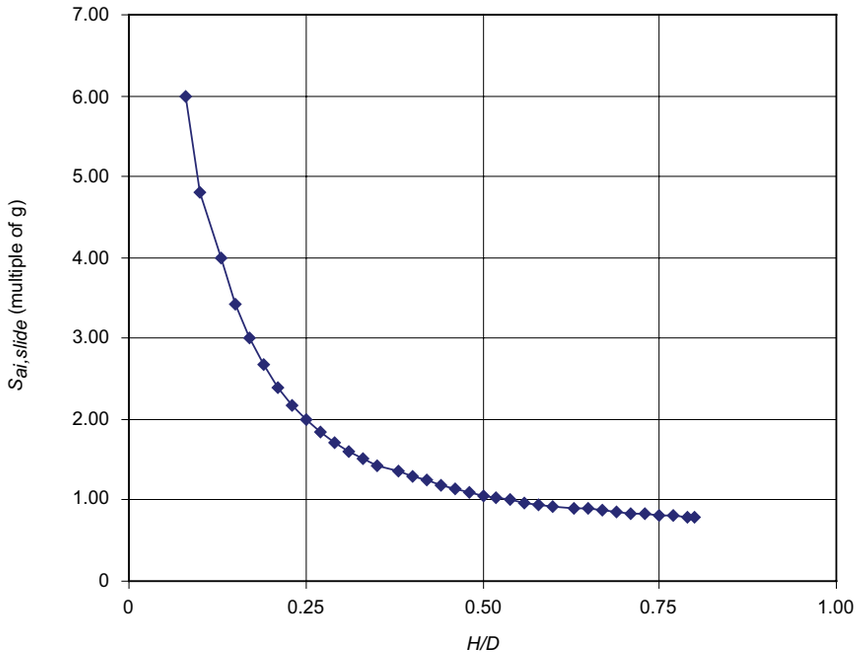


Figure A2 Design spectral response acceleration that causes the tank to slide $S_{ai,slide}$

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The sliding resistance in Eq A14-3 is based on a static coefficient of friction equal to tan 30°. The coefficient tan 30° is a best estimate for bottom plates placed on concrete or cushions constructed of sand, crushed rock, or asphaltic road mix. A plot of Eq A14-3 is shown in Figure A2.

Sec. A.14.3 Seismic Design Loads

A.14.3.1 *Natural periods.* For the site-specific procedure, the natural period of the shell-fluid system may be determined using API 650 or reference 10, 12, or 13 of Sec. A.2 of Appendix A.

The first mode sloshing wave period T_c may be determined by Eq 14-14 or by the graphical procedure utilizing Eq A14-4 and Figure A3.

$$T_c = K_p \sqrt{D} \qquad \text{(Eq A14-4)}$$

where:

T_c = first mode sloshing wave period, in seconds
K_p = factor from Figure A3 for the ratio of *D/H*
D = tank diameter, in feet
H = distance from bottom of shell to MOL, in feet

A.14.3.2 *Design overturning moment at the bottom of the shell.* The standard provides an equation (Eq 14-15) for determining the overturning moment at the bottom of the shell M_s. The overturning moment M_s is used in the design of the shell and anchorage, and does not depend on the type of foundation.

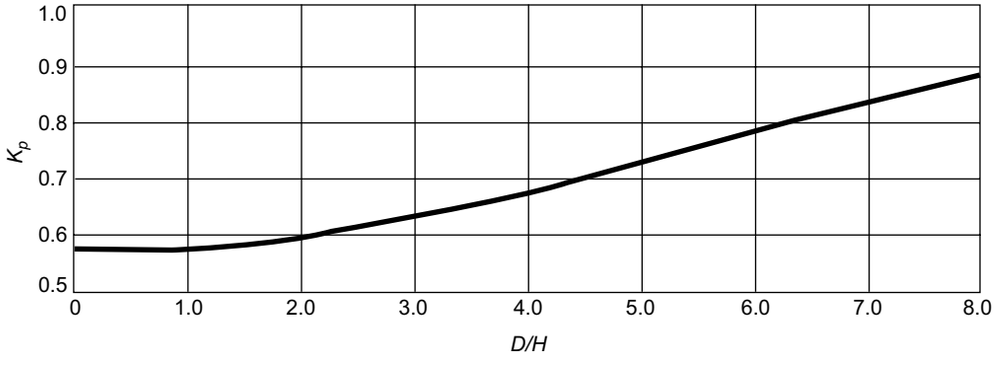


Figure A3 Curve for obtaining factor K_p for the ratio *D/H*

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A.14.3.2.2 Effective weight of tank contents. The effective impulsive and convective weights W_i and W_c may be determined by equations Eq 14-16 through Eq 14-18, or by multiplying W_T by the ratios W_i/W_T and W_c/W_T obtained from Figure A4. The heights X_i and X_c from the bottom of the shell to centroids of the lateral seismic forces applied to W_i and W_c may be determined by equations Eq 14-20

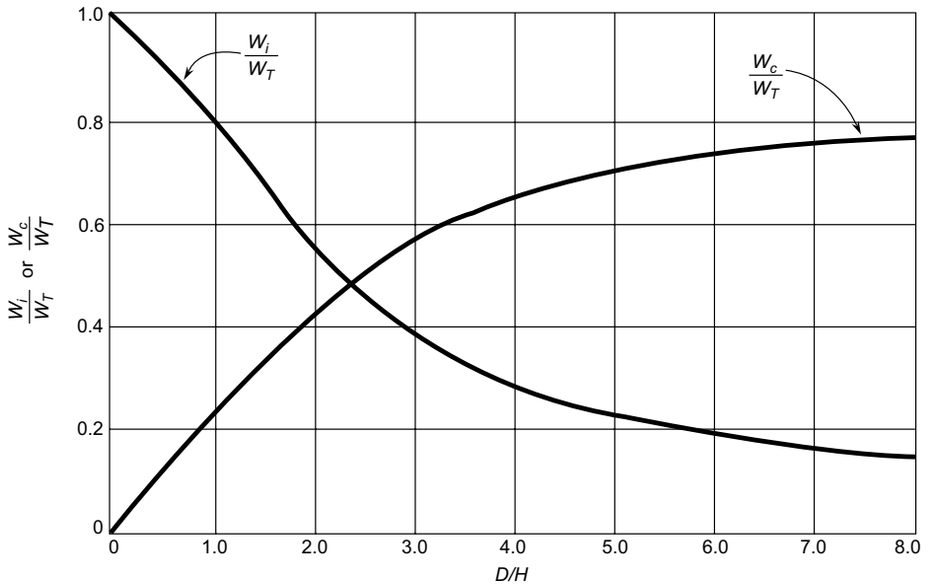


Figure A4 Curves for obtaining factors W_i/W_T and W_c/W_T for the ratio D/H

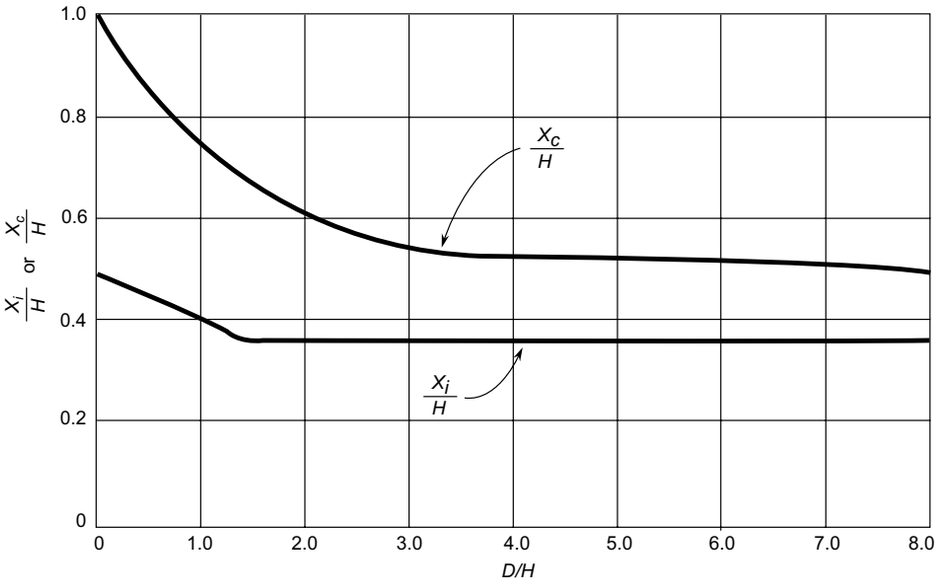


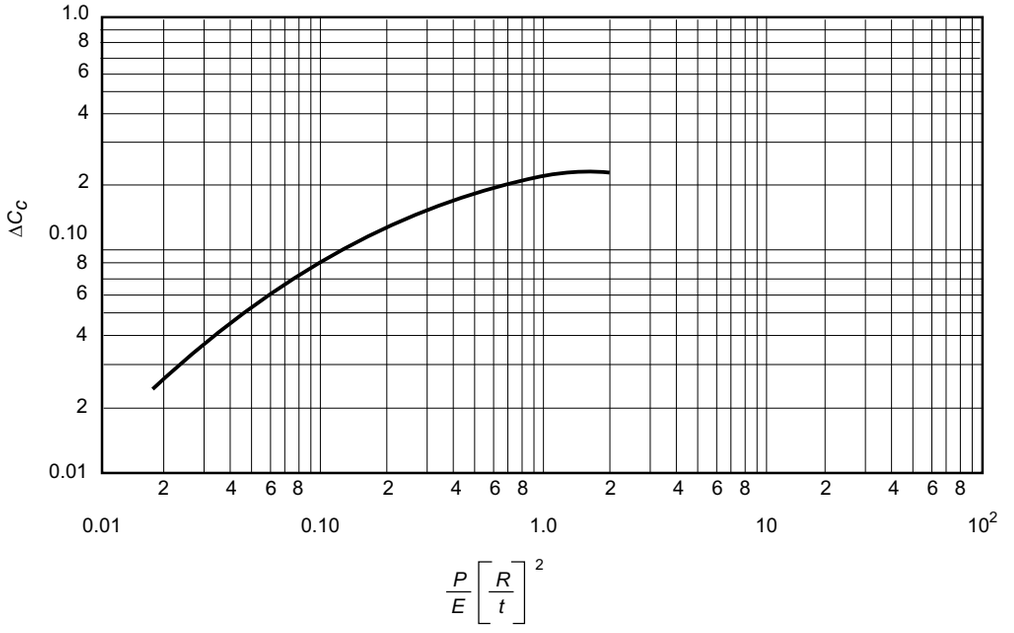
Figure A5 Curves for obtaining factors X_i/H and X_c/H for the ratio D/H

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through Eq 14-22, or by multiplying H by the ratios X_i/H and X_c/H obtained from Figure A5.

A.14.3.3 *Design shear and overturning moment at the top of the foundation.* The standard provides equations for determining the shear V_f and overturning moment at the top of the foundation. For tanks supported by ringwall or berm foundations, the overturning moment at the top of the foundation equals the moment at the bottom of the shell M_s (Eq 14-15). For tanks supported by mat or pile cap foundations (i.e., mat or cap under the entire tank), the overturning moment at the top of the foundation M_{mf} equals the overturning moment at the bottom of the shell M_s plus the moment due to varying bottom pressures on the mat or pile cap. The equation for overturning moment M_{mf} (Eq 14-24) is based on centroid heights that have been modified to include the effects of varying bottom pressures. The modified centroid heights for the impulsive and convective components X_{imf} and X_{cmf} are shown in equations Eq 14-25 through Eq 14-27.

A.14.3.4.2.4 Allowable shell stresses. The pressure-stabilizing buckling coefficient ΔC_c used to determine the seismic allowable



ΔC_c = *pressure stabilizing buckling coefficient*
P = hydrostatic pressure at point under consideration in pounds per square inch
E = the modulus of elasticity
R = the radius of tank, in inches
t = the thickness of the plate under consideration

Figure A6 Pressure-stabilizing buckling coefficient ΔC_c for self-anchored tanks

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longitudinal shell compression stress may be determined by Eq 14-42 and Eq 14-43, or it can be obtained from Figure A6.

A.14.3.4.3 Vertical design acceleration. Several of the equations for ground-supported flat-bottom tanks combine the effects from horizontal and vertical design accelerations by the SRSS method. Where no equation is provided, the effects from horizontal and vertical design accelerations may be combined by the direct sum method, with the load effects from vertical design acceleration being multiplied by 0.40, or the SRSS method.

Sec. A.14.4 Freeboard

If the freeboard for the sloshing wave is not included in the design, the seismic overturning moment and the design of the roof and supporting structure shall be designed for the effects of the sloshing wave as specified in Sec. 14.4. The purchaser should take this into consideration when determining the MWL. (See foreword Sec. III.A.4.a.)

Sec. A.14.5 Piping Connections

A.14.5.1 *Flexibility.* The maximum uplift at the base of the shell for ground-supported flat-bottom tanks that are self-anchored may be approximated by the equation

$$y_u = \frac{F_y L^2}{83{,}300 t_b} \qquad \text{(Eq A14-5)}$$

where:

y_u = maximum uplift at the base of the shell, in inches
F_y = minimum specified yield strength of the bottom annulus, in pounds per square inch
L = required width of the bottom annulus measured from the inside of the shell, in feet, determined by Eq 14-30
t_b = design thickness of the bottom annulus, in inches

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SECTION A.15: WIND DESIGN OF FLAT-BOTTOM WATER STORAGE TANKS (REFER TO SECTION 15 OF ANSI/AWWA D103)

Sec. A.15.1 Wind Pressure

In this standard, ASCE 7-10 is primarily used as the basis for the minimum wind design loads. In Eq 15-2, the calculated wind pressures are limited to a minimum of $30C_f$, which is consistent with the minimum wind pressures specified in previous editions of ANSI/AWWA D103.

ASCE 7 includes a topographic factor K_{zt} to account for wind escalation over hills, ridges, and escarpments. ANSI/AWWA D103 assumes a value of 1.0 for K_{zt}. If site conditions are such that the topographic effects should be considered, the user is referred to ASCE 7 for guidelines in defining K_{zt}.

ANSI/AWWA D103 assumes a directionality factor K_d of 1.0. This is appropriate for tanks with equal lateral load resistance in all directions. It should be noted that ASCE 7 has increased the load factor on wind from 1.3 to 1.6 for ultimate strength design. This was done in part because a wind directionality factor of 0.85 was built into the 1.3 load factor. By assuming that K_d = 1.0, the calculated wind loads may be used with load factors that have not been adjusted upward to account for the removal of this factor.

The gust effect factor G accounts for along-wind loading effects due to wind turbulence-structure interaction. This standard uses a gust effect factor of 0.85 min. The gust factor is not intended to account for across-wind loading effects, vortex shedding, instability due to galloping, or flutter of dynamic torsional effects. Tall, slender tanks should also be evaluated for these effects using the recognized literature referenced in ASCE 7.

Sec. A.15.2 Anchor Check

Loads associated with the minimum operating level of water may be used to resist uplift forces for ground-supported flat-bottom tanks.

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The minimum operating level is usually the invert of the inlet/outlet pipe for ground-supported flat-bottom tanks.

Sec. A.15.3 Sliding Check

Where necessary, anchorage shall be provided to resist horizontal base shear from wind or seismic loads. Such anchorage shall be designed for shear transfer using shear friction.

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