ANSI/AWWA **D100-21**

(Revision of ANSI/AWWA D100-11)

AWWA Standard

Welded Carbon Steel Tanks for Water Storage

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American Water Works Association

Approved American National Standard ANSI

STANDARD Since 1881

AWWA Standard

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Foreword

This foreword is for information only and is not a part of ANSI/AWWA D100.*

I. Introduction.

I.A. *Background.* In 1931, American Water Works Association (AWWA) Subcommittee 7H, whose members were L.R. Howson, H.C. Boardman, and James O. Jackson, prepared "Standard Specifications for Riveted Steel Elevated Tanks and Standpipes." The specifications were published in the November 1935 edition of *Journal AWWA.* In 1940, the scope of the standard specifications was expanded to include welded construction. The American Welding Society (AWS)† cooperated in the revision and became a joint sponsor of the standard. Since its original publication, the standard has gained wide acceptance in the United States and abroad.

I.B. *History.* In 1965, Appendix C was added to provide for the alternative use of higher-strength steels for standpipes and reservoirs. Other changes included the addition of requirements for the use of steel pipe as tubular columns and a wind–pressure formula for winds in excess of 100 mph (45 m/s). The requirements for loads on balconies and ladders and unit stresses for combinations of wind, seismic, and other loads were clarified. The rules for the minimum thickness of shell plates for standpipes and reservoirs were revised to apply only to cylindrical shells and not to roof knuckles or toroidal or elliptical roof plates containing water. The swivel ladder for standpipes and reservoirs, which was found to be impractical, was eliminated, and a fixed ladder was required. The rules for welding and for weld qualification were rewritten completely. The qualification procedure of the American Society of Mechanical Engineers (ASME)‡ Boiler and Pressure Vessel Code, Sec. IX, was adopted, and the sizes of fillet welds in the shell-to-bottom joints of standpipes and reservoirs were revised, as were the sections on sand cushions and grouting for standpipe and reservoir bottoms. Rules for inspection of welds were rewritten completely. An isothermal map showing the lowest one-day mean temperature in various parts of the continental United States and parts of Canada was included. Concrete foundation design was brought into conformity with American Concrete Institute (ACI)§ Standard No. 318, Building Code Requirements for Reinforced Concrete.

* American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036.
† American Welding Society, 8669 NW 36 Street, Suite 130, Miami, FL 33166.
‡ The American Society of Mechanical Engineers, Two Park Avenue, New York, NY 10016.
§ American Concrete Institute, 38800 Country Club Drive, Farmington Hills, MI 48331.

In 1973, the use of rivets for joints in tank shells was eliminated. Specifications for tank steels were revised to include low-alloy steels. The design of foundations for elevated tanks and standpipes was changed extensively, making foundation design a part of the requirements. Procedures for soil investigation were recommended.

In 1979, Appendix A, Non-Mandatory Seismic Design of Water Storage Tanks, and Appendix B, Diagrams for Checking Overturning of Elevated Tanks, were added. The sections from the former Appendix B, covering information to be provided, were incorporated into Sec. II of the foreword, and the sections dealing with foundations were incorporated into Sec. 12. Sec. 11 was revised to include inspection and testing requirements that were formerly in Sec. 11 and Sec. 12 and appendixes A and B. Other additions included requirements for additional acceptable steels, design requirements for seismic resistance, a formula for cylindrical shell design, requirements for backfill within ringwall foundations, and requirements for depth-of-pipe cover. The out-of-date porosity charts in former Appendix A were eliminated and reference made to the charts in the ASME Boiler and Pressure Vessel Code, Sec. VIII, or to the identical charts in American Petroleum Institute (API)[¶] Standard 650, Welded Tanks for Oil Storage. A section covering permissible inspection by air carbon arc gouging was added to Sec. 11. Materials for shell plates and intermediate stiffeners were classified into three categories in Appendix C, and the requirements for impact testing were expanded.

In 1984, revisions included new sections pertaining to single-pedestal tanks incorporating design rules for this type of tank. New design rules were included for columns of elevated tanks having eccentric work-point connections. A section covering the design considerations for struts was added. For combined stresses, the unit stresses for wind and seismic forces were increased from 25 percent to 33⅓ percent. Shell plates thicker than 2 in. (51 mm), conforming to ASTM** A36, Specification for Structural Steel, were allowed to be used, provided that their usage was in compliance with certain stipulated conditions and requirements. Ground-supported tanks not greater than 50 ft (15.2 m) in diameter were allowed to have a minimum shell thickness of 3⁄16 in. (4.8 mm). A minimum size and maximum spacing were added for foundation bolts. The previous Appendix A, on seismic design, was incorporated into the standard as Sec. 13. In addition, a new section was added to Sec. 13 to permit scaling down to specific site response spectra when local seismic data are available.

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

** ASTM International, 100 Barr Harbor Drive, P.O. Box C700, West Conshohocken, PA 19428-2959.

Appendix C, Alternative Rules and Design Stresses for the Use of Steel Plates and Shapes with Suitable Toughness and Ductility for Use in Welded Standpipes and Reservoirs at Specified Minimum Ambient Temperatures, was made a part of the standard while retaining its title designation as Appendix C.

For Appendix C tanks with a height-to-diameter (*H/D*) ratio of 0.50 or less, the shell design was allowed to be by the Variable Design Point Method, in compliance with API 650. Also, for Appendix C tanks, inspection of certain members is not required when the material has a tensile strength less than 75,000 psi (517.1 MPa).

In 1996, revisions included new requirements for high-strength anchor bolts. Table 1 was added to clarify thickness limitations and special material requirements. Requirements for wind escalation for heights greater than 125 ft (38.1 m) and wind loads on shrouds were added. Fixed-percentage seismic design loads were eliminated. Design requirements for handrails and guardrails were added. Allowable-unit stresses were stated as a function of material class, which is a function of material yield strength. Width-to-thickness limitations were added for compression elements, and compression requirements for shells were clarified. Design rules for tension and compression rings were added. Anchorage requirements were expanded, and a wind overturning check for ground-supported tanks was added. Weld inspection for tension bracing for cross-braced, column-supported elevated tanks was expanded to include ultrasonic testing and tensile tests. Requirements for flush-type cleanout fittings for ground-supported flat-bottom tanks were added. Design rules and limits for openings in support pedestals were added. Criteria for accessories including safety grills, overflows, and vents were updated. Seal welds were defined and usage clarified. Temperature requirements for welding and weld reinforcement limits were added. Tolerances were added for ground-supported tanks and shells designed by stability formulas. Responsibilities of the certified welding inspector were defined. Inspection requirements for primary and secondary stressed joints and tubular support columns were clarified. Inspection requirements were added for single-pedestal columns and large-diameter dry risers. The penetrometer techniques and details were revised to conform to ASME criteria.

The load factor to be applied to water load for foundation design was clarified, and requirements for material under bottom plates of ground-supported tanks were added.

Seismic design load equations were revised to follow the Uniform Building Code[††] format. A new seismic map of the United States was included along with new and revised equations for calculating such things as hydrodynamic seismic hoop tensile

[††] Uniform Building Code, International Conference of Building Officials, 5360 Workman Mill Road, Whittier, CA 90601.

stresses and sloshing wave height to determine minimum freeboard for ground-supported flat-bottom tanks.

Appendix C of the previous edition was incorporated in the standard as Sec. 14, and reference standards were moved to Sec. 1. Electrode criteria and requirements for permanent and temporary attachment criteria were revised. The type of inspection and number of weld-joint inspections were updated to improve quality control.

A new Sec. 15, entitled Structurally Supported Aluminum Dome Roofs, was added.

In 2005, the title and scope of the standard were revised to address only new tanks constructed of welded carbon steel that are used to store water at atmospheric pressure. All contractual language was removed, and nonmandatory requirements were moved to Appendix A as commentary. Specific editions were added for all references. Wind loads were revised to align with ASCE[‡‡] 7-05. Two new methods (Methods 2 and 3) for determining the allowable local buckling compressive stress for shells were added. Method 3 permits an increase in the allowable stress due to pressure stabilization and is based on a nonlinear buckling analysis. Method 2 permits a partial increase in the allowable stress due to pressure stabilization. The existing method for determining the allowable local buckling compressive stress was renamed Method 1. For roof rafters designed for roof live load of 50 lb/ft^2 or less, allowable stresses were limited to those of ASTM A36 material. For roof rafters designed for roof live load greater than 50 lb/ft^2, higher allowable stresses may be used when using a material with minimum specified yield strength greater than ASTM A36 material. Extensive requirements were added for anchor bolts and anchor straps. The thickness to which corrosion allowance is added was changed to the thickness determined by design for elements other than bottom plate of ground-supported flat-bottom tanks. A minimum width requirement was added for butt-welded annulus plates. The requirement that welded splices in tension bracing for multicolumn tanks must be designed for 100 percent joint efficiency was clarified. The ⅟₁₆-in. (1.6-mm) additional shell thickness requirement for flush-type cleanouts was eliminated to match the current requirements of API 650. The requirement that inlet protection be removable was added for elevated tanks. Electrical isolation requirements were added for dissimilar metals inside the tank below the MWL.

Sec. 6 (AWWA D100-96), titled Sizing of Ground-Supported Standpipes and Reservoirs, was deleted.

The full-size proof test requirement for the qualification of welding procedure specifications for tension-bracing splice welds was increased to ⅘ times the published

[‡‡] American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, VA 20191.

minimum yield strength of the bracing member. Minimum fillet weld size requirements relative to root opening were clarified, and a maximum root opening requirement (3⁄16 in. [4.76 mm]) was added. Seal-welding requirements for corrosion protection and preheat requirements were clarified. Inspection based on sectional segments was deleted. The requirement that welds be visually inspected and acceptance criteria were added. Measurement and documentation requirements for shells designed by Method 2 or 3 were added. Qualification of welder and production testing requirements were added for tension-bracing splice welds. The proof test for tension-bracing splice welds was increased to 4⁄3 times the published minimum yield strength of the bracing material.

For foundations, a one-third increase in the allowable bearing stress for wind loads when specified in the geotechnical report was added.

Seismic loads were revised to align with the seismic load requirements of FEMA[§§] 450 and proposed ASCE 7-05, which are based on a maximum considered earthquake ground motion for an event with a 2 percent probability of exceedance within a 50-year period (recurrence interval of approximately 2,500 years). General and site-specific procedures for determining design response spectra were included. Alternate procedures for elevated tanks and ground-supported flat-bottom tanks were added and allow the use of soil-structure and fluid-structure interaction. The requirement that P-delta effects be considered was added for all elevated tank styles. Vertical design acceleration requirements were specified and are now mandatory for all tanks. A critical buckling check for pedestal-type elevated tanks was added to guard against premature buckling failure. Equations were added to calculate the overturning moment for mat or pile cap foundations supporting flat-bottom tanks. Minimum freeboard requirements similar to those of ASCE 7-05 were added for ground-supported flat-bottom tanks. Piping flexibility requirements similar to those of ASCE 7-05 were added for all tanks.

Appendix A, Commentary for Welded Carbon Steel Tanks for Water Storage, was added to provide background information for many of the requirements contained in the standard. Recommendations for antennas and related equipment were included.

Appendix B, Default Checklist, was added to assist users of the standard.

In 2011, revisions included corrections, updates, and new material to clarify some of the existing requirements.

Sec. 1 was revised to show the latest edition of references.

Sec. 3 was revised to match the wind exposure definitions of ASCE 7-05. The requirement that dome roofs constructed of aluminum shall comply with ANSI/

[§§] Building Seismic Safety Council, 1090 Vermont Avenue, NW, Suite 700, Washington, DC 20005.

AWWA D108 was added. The stress evaluation procedure of ASME BPVC Sec. VIII, Div. 2, was added as an acceptable method for evaluating local shell or pedestal stress for anchor chair designs that are based on a detailed analysis.

Sec. 10 was revised to include an erection tolerance multiplier for elements of ground-supported flat-bottom tanks that are designed in accordance with Sec. 3.4.3 and subject to small compressive stresses.

Sec. 11 was revised to make leak testing of the bottom-to-shell joint mandatory for all ground-supported flat-bottom tanks.

Sec. 13 was revised to not require a site response analysis for short-period structure located on liquefiable soils. The site-specific procedure of FEMA 450 was deleted, and the site-specific procedures of ASCE 7-05 were referenced. The scaling requirement for the alternate procedures was clarified.

Sec. 14 was revised to clarify the thickness and design metal temperature requirements for Category 1 and Category 2 materials when impacts are provided.

Sec. 15, covering dome roofs constructed of aluminum, was deleted and replaced with a reference to ANSI/AWWA D108.

The major revisions in this edition are summarized in Sec. IV of this foreword.

The last edition was approved by the AWWA Board of Directors on Jan. 23, 2011. This edition was approved on Jan. 25, 2021.

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

In the United States, authority to regulate products for use in, or in contact with, drinking water rests with individual states.¶ 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.

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

2. Two standards developed under the direction of NSF, NSF***/ANSI/CAN††† 60, Drinking Water Treatment Chemicals—Health Effects, and NSF/ANSI/CAN 61, Drinking Water System Components—Health Effects.

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/CAN 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/CAN 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 D100 does not address additives requirements. Users of this standard should consult the appropriate state or local agency having jurisdiction in order to

1. Determine additives requirements, including applicable standards.

2. Determine the status of certifications by parties offering to certify products for contact with, or treatment of, drinking water.

3. Determine current information on product certification.

II. Special Issues. This standard has no applicable information for this section.

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.

Contractual responsibilities for items such as design, material, fabrication, construction, inspection, and testing need to be addressed by the purchaser.

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

††† Standards Council of Canada, 55 Metcalfe Street, Suite 600, Ottawa, ON K1P 6L5 Canada.

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

This standard is based on the accumulated knowledge and experience of purchasers and manufacturers of welded steel tanks.*

Many tanks built in compliance with the first edition of this standard are more than 75 years old and are still in service. Properly operated and maintained welded steel water tanks can have an almost unlimited service life.

III.A. *Purchaser Options and Alternatives*. Proper use of this standard requires that the purchaser 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. This standard is not intended to cover storage tanks that are to be erected in areas subject to regulations that are more stringent than the requirements contained herein. In such cases, local regulations supersede the requirements of this standard. Where local, municipal, county, or state government requirements exist, such requirements are to govern, and this standard should be interpreted to supplement them. It is the purchaser's responsibility to supplement or modify this standard for compliance with these local requirements. In addition, the purchaser is to provide clarification of the governing codes where they do not clearly refer to tanks, but where the purchaser intends such stipulations to apply to the tank under contract. As an example, if a governing code stipulates a building roof snow load of 40 lb/ft^2 (1,915 N/m^2) and it is intended that the tank roof be designed for this load, the purchaser is to include this as a clarification.

The details of design and construction covered by this standard are minimum requirements. At a minimum, it is important that all of the design conditions in this standard be met.† A tank cannot be represented as an ANSI/AWWA D100 tank if it does not meet the minimum requirements of this standard.

The foundations of tanks are one of the more important aspects of tank design; detailed requirements are covered in Sec. 12. The purchaser should obtain an adequate soil investigation at the site, including recommendation of the type of foundation to be used, the depth of foundation required, and the design soil-bearing pressure. This information should be established by a qualified geotechnical engineer.

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* The word "tanks" is used hereinafter broadly in place of the lengthy phrase "elevated tanks, standpipes, and reservoirs for water storage."

† Dawe, J.L., C.K. Seah, and A.K. Abdel-Zaher. 1993. Investigation of the Regent Street Water Tower Collapse. *Journal AWWA,* 93(5):34–47.

A drainage-inlet structure or suitable erosion protection should be provided to receive discharge from the tank overflow. The overflow should not be connected directly to a sewer or a storm drain without an air break.

Annual inspection and maintenance of the exposed side of the tank shell-to-bottom connection for a standpipe or reservoir is important if maximum tank life is to be attained. In particular, accumulations of dirt and weeds, which may trap moisture and accelerate corrosion, should be removed. Inspection of the interior and exterior of the entire tank with corrective maintenance at three-year intervals is recommended. Refer to AWWA Manual M42, *Steel Water-Storage Tanks,* for guidance concerning inspection and maintenance of welded steel tanks for water storage.

This standard assumes that the purchaser (owner) provides sufficient water replacement and circulation to prevent freezing in the tank and riser pipe. Where low usage may result in the possibility of freezing, water may need to be wasted or heat provided to prevent freezing. The purchaser is referred to National Fire Protection Association (NFPA)[‡] document NFPA 22, Standard for Water Tanks for Private Fire Protection, for heater sizing. Purchasers are cautioned against allowing ice buildup for insulation, which may break loose and damage the tank.

This standard does not cover tank disinfection procedures or cleaning and painting. ANSI/AWWA C652, Disinfection of Water-Storage Facilities, should be consulted for recommended procedures for disinfection of water storage facilities. Often, it is desirable for the purchaser to perform the disinfection to eliminate the necessity for the painting constructor to return afterward or to stand by until the inside paint has dried completely. If disinfection is to be done by either the tank or painting constructor, the purchaser must specify the manner in which disinfection is to be done.

The following recommendations are believed to represent good practice, but they are not requirements of ANSI/AWWA D100. When a welded steel tank is to be purchased under this standard, the purchaser should provide the following:

1. The site on which the tank is to be built, including sufficient space to permit the structure to be erected by customary methods.

2. Water at the proper pressure for testing, as required, and facilities for disposal of wastewater after testing.

3. A suitable right-of-way from the nearest public road to the erection site.

4. Materials furnished by the purchaser to be used by the constructor for construction of the tank.

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

5. A geotechnical investigation of the project site that provides the information listed in Sec. 12.2.1.

The constructor should provide the following items:

1. Foundation and tank design, drawings, and specifications.
2. All labor and materials, except materials provided by the purchaser, necessary to complete the structure, including the foundations, accessories, and testing required by this standard.
3. Any additional work, separately specified by the purchaser, such as painting and disinfection.

Variations in the responsibilities of both the purchaser and the constructor, as previously outlined, may be made by contractual agreement. The purchaser and the bidder should each provide the information identified in the following listings.

III.A.1. Information to Be Provided by Purchaser for an Elevated Tank. This standard provides minimum requirements for the design, construction, inspection, and testing of the tank without any designation of which party must perform these tasks. For this reason, the following information should be provided by the purchaser:

1. The standard to be used—that is, ANSI/AWWA D100, Welded Carbon Steel Tanks for Water Storage, of latest revision.
2. Whether compliance with NSF/ANSI/CAN 61, Drinking Water System Components—Health Effects, is required.
3. Capacity.
4. Bottom capacity level (BCL) or maximum operating level (MOL) above top of foundation.
5. Type of roof.
6. Head range, if specific range is required.
7. Diameter and type of riser.
8. Location of site.
9. Desired time for completion.
10. Name of, and distance to, nearest town.
11. Name of, and distance to, nearest railroad siding.
12. Type of road available for access to the site and whether it is public or private.
13. Availability of electric power; who furnishes it; at what fee, if any; what voltage; whether direct or alternating current; and, if alternating current, what cycle and phase.
14. Availability of compressed air, pressure, volume, and fee, if any.
15. Whether details of all welded joints are to be provided (Sec. 1.3).

16. Whether mill test reports are required (Sec. 2.1).

17. Details of other federal, state, local, and provincial requirements.

18. Type of pipe and fittings for fluid conductors (Sec. 2.2.11), including type of pipe joint if different from that permitted in Sec. 2.2.11.

19. The risk category for the structure (Sec. 3.1.1).

20. If the tank is located in an area designated CS in or at an elevation above the limits of ASCE 7, Figure 7-1, the ground snow load (Sec. 3.1.4.1).

21. If tank is located in a special wind region, the basic wind speed (Sec. 3.1.6.1.1).

22. Corrosion allowance, if any, to be added to parts that will be in contact with water and to parts that will not be in contact with water (Sec. 3.9).

23. Whether a balcony is required for inspection and painting when a horizontal girder is not required by the tank design (Sec. 4.4.4.2).

24. Location of manholes, ladders, and any additional accessories required (Sec. 5).

25. Number and location of pipe connections, and type and size of pipe to be accommodated.

26. Whether a safety grill at the top of the riser is required (Sec. 5.1.1).

27. Whether a removable silt stop is required (Sec. 5.2.1).

28. Overflow type, whether stub, to ground, or (if applicable) to extend below balcony; size of pipe; and pumping and discharge rates (Sec. 5.3).

29. Whether personal fall arrest systems, rest platforms, roof–ladder guardrails, or other safety devices are required and on which ladders, and whether requirements in excess of OSHA[§] CFR Part 1910 are required (Sec. 5.4). NOTE: Purchaser is to specify beginning location of outside tank ladder if other than at a level of 10 ft (3.0 m) above grade (Sec. 5.4.2.2).

30. Whether a special pressure-vacuum-screened vent or a pressure-vacuum relief mechanism is required for the tank vent (Sec. 5.5.2).

31. Requirements for any additional accessories required, including provisions for antennas and related equipment (Sec. 5.6).

32. Whether welding procedure specifications are to be provided (Sec. 8.2.1.5).

33. For butt-joint welds subject to secondary stress, whether complete joint penetration is to be provided at joints in base metals of thicknesses greater than ⅜ in. (9.5 mm) (Sec. 8.4.2 (2)).

34. Whether seal welding is required and, if so, where it is required (Sec. 8.14.2).

[§] Occupational Safety and Health Administration, 200 Constitution Avenue N.W., Washington, DC 20210.

35. Whether the purchaser will provide shop inspection.

36. Whether a written report is required certifying that the work was inspected as set forth in Sec. 11.2.

37. Whether radiographic film and inspection reports must be provided (Sec. 11.2).

38. Kinds of paint or protective coatings and number of coats for inside and outside surfaces (see ANSI/AWWA D102, Coating Steel Water-Storage Tanks).

39. Soil investigation (Sec. 12.2.1), including foundation design criteria, type of foundation, depth of foundation below existing grade, Site Class for seismic areas, and design soil-bearing pressure, including factor of safety (Sec. 12.3). NOTE: Unless otherwise specified, the top of foundation(s) shall be a minimum of 6 in. (152 mm) above finish grade (Sec. 12.7.1).

40. Pile type and depth below existing grade when a pile-supported foundation is required (Sec. 12.7.3) and provisions for establishing criteria for compensation adjustment due to piling length changes resulting from varying subsurface conditions.

41. Whether the effect of buoyancy is to be considered in the foundation design (Sec. 12.7.4).

42. Whether requirements of ACI 301, Specifications for Structural Concrete, are applicable to the concrete work (Sec. 12.8).

43. Vertical distance from finished ground level to the crown of inlet and outlet pipe (earth cover) at riser pier (Sec. 12.9.2), if different from Figure 3.

44. Whether the site-specific procedure of Sec. 13.2.7 is required.

45. Whether third-party inspection will be used by the purchaser and for which items.

III.A.2. Information to Be Provided by Purchaser for a Standpipe or Reservoir (Ground-Supported Flat-Bottom Tanks). This standard provides minimum requirements for the design, construction, inspection, and testing of the tank without any designation of which party must perform these tasks. For this reason, the following items should be provided by the purchaser:

1. The standard to be used—that is, ANSI/AWWA D100, Welded Carbon Steel Tanks for Water Storage, of latest revision.

2. Whether compliance with NSF/ANSI/CAN 61, Drinking Water System Components—Health Effects, is required.

3. Capacity.

4. Maximum operating level (MOL) above top of foundation.

5. Type of roof.

6. Location of site.

7. Desired time for completion.

8. Name of, and distance to, nearest town.

9. Name of, and distance to, nearest railroad siding.

10. Type of road available for access to the site and whether it is public or private.

11. Availability of electric power; who furnishes it; at what fee, if any; what voltage; whether direct or alternating current; and, if alternating current, what cycle and phase.

12. Availability of compressed air, pressure, volume, and fee, if any.

13. Whether details of all welded joints are to be provided (Sec. 1.3).

14. Whether mill test reports are required (Sec. 2.1).

15. Details of other federal, state or provincial, and local requirements.

16. Type of pipe and fittings for fluid conductors (Sec. 2.2.11), including type of pipe joint if different from that permitted in Sec. 2.2.11.

17. The risk category for the structure (Sec. 3.1.1).

18. If the tank is located in an area designated CS in or at an elevation above the limits of ASCE 7, Figure 7-1, the ground snow load (Sec. 3.1.4.1).

19. If tank is located in a special wind region, the basic wind velocity (Sec. 3.1.6.1.1).

20. Corrosion allowance, if any, to be added to parts that will be in contact with water and to parts that will not be in contact with water (Sec. 3.9). This also applies when a tank is to comply with Sec. 14.

21. Size and quantity of flush-type cleanouts, if required (Sec. 3.13.2.5).

22. Location of manholes, ladders, and additional accessories required (Sec. 7).

23. Number and location of pipe connections, and type and size of pipe to be accommodated.

24. The bottom capacity level (BCL) of the tank, when empty, if it differs from the level when the tank would be emptied through the specified discharge fittings (Sec. 7.2).

25. Whether a removable silt stop is required (Sec. 7.2.1).

26. Overflow type, whether stub or to ground; size of pipe; and pumping and discharge rates (Sec. 7.3).

27. Whether personal fall arrest systems, rest platforms, roof–ladder guardrails, or other safety devices are required and on which ladders, and whether requirements in excess of OSHA CFR Part 1910 are required (Sec. 7.4). Note: Purchaser is to specify

beginning location of outside tank ladder if other than at a level of 10 ft (3.0 m) above the level of the tank bottom (Sec. 7.4.2.2).

28. Whether a special pressure-vacuum-screened vent or a pressure-vacuum relief mechanism is required for the tank vent (Sec. 7.5.2).

29. Requirements for any additional accessories required, including provisions for antennas and related equipment (Sec. 7.6).

30. Whether welding procedure specifications are to be furnished (Sec. 8.2.1.5).

31. For butt-joint welds subject to secondary stress, whether complete joint penetration is to be provided at joints in materials of thicknesses greater than ⅜ in. (9.5 mm) (Sec. 8.4.2 (2)). Note: For tanks that are to comply with Sec. 14, complete joint penetration is required for all butt-welded shell joints.

32. Whether seal welding is required and, if so, where it is required (Sec. 8.14.2).

33. Whether the purchaser will provide shop inspection.

34. Whether a written report is required certifying that the work was inspected as set forth in Sec. 11.2.

35. Whether radiographic film and inspection reports must be provided (Sec. 11.2).

36. Kinds of paint or protective coatings and number of coats required for inside and outside surfaces except underside of bottom (see ANSI/AWWA D102, Coating Steel Water Storage Tanks).

37. Soil investigation (Sec. 12.2.1), including foundation design criteria, type of foundation (Sec. 12.6), depth of foundation below existing grade, Site Class for seismic areas, and design soil-bearing pressure, including factor of safety (Sec. 12.3). Note: Unless otherwise specified, the top of the foundation is to be a minimum of 6 in. (152 mm) above the finish grade (Sec. 12.7.1).

38. Pile type and depth below existing grade when a pile-supported foundation is required (Sec. 12.7.3). The provisions for establishing criteria for compensation adjustment due to piling length changes resulting from varying subsurface conditions.

39. Whether the effect of buoyancy is to be considered in the foundation design (Sec. 12.7.4).

40. Whether requirements of ACI 301, Specifications for Structural Concrete, are applicable to the concrete work (Sec. 12.8).

41. Vertical distance from finished ground level to the crown of inlet and outlet pipes (earth cover) at tank foundation (Sec. 12.9.2), if different from Figure 3.

42. Whether the site-specific procedure of Sec. 13.2.7 is required.

43. Whether seismic design of roof framing and columns is required (Sec. 13.5.4.4) and amount of live loads to be used.

44. Whether design in accordance with Sec. 14 is allowed or required (Sec. 14.1.1). For tanks designed under Sec. 14, specify the design metal temperature (Sec. 14.2.4).

45. Whether a certified welding inspector is required for Sec. 14 tanks (Sec. 14.4.5).

46. Whether third-party inspection will be used by the purchaser and for which items.

III.B. *Information to Be Provided with Bid.*

III.B.1. Information to Be Provided with the Bid for an Elevated Tank. The following information should be provided by the bidder for an elevated tank:

1. A drawing showing the dimensions of the tank and tower, including the tank diameter, the height to BCL and MOL, sizes of principal members, and thickness of plates in all parts of the tank and tower. Also, the maximum wind or seismic gross moment and shear on the foundation system should be identified.
2. The number, names, and sizes of all accessories.
3. Painting information, if included.

III.B.2. Information to Be Provided with the Bid for a Standpipe or Reservoir (Ground-Supported Flat-Bottom Tanks). The following information shall be provided for a ground-supported flat-bottom tank:

1. A drawing of the standpipe or reservoir showing
   a. design basis (i.e., whether Sec. 14 is used);
   b. diameter, height to the MOL, and shell height;
   c. shell plate widths, thicknesses, and grades;
   d. roof type, thickness, and the type, size, and configuration of roof support structure (if any);
   e. bottom thickness;
   f. thickness, width, and grade of butt-welded annulus (if any); and
   g. type, size, and quantity of mechanical anchors (if any).
2. The number, names, and sizes of all accessories.
3. Painting information, if included.

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

IV. Major Revisions. This edition of the standard includes numerous corrections, updates, and new material to clarify some of the existing requirements.

1. Sec. 1 was revised to show the latest edition of references.

2. Sec. 2 was revised to eliminate obsolete material references (i.e., ASTM A283/A283M, grades A and B).

3. Sec. 3 was revised to include risk category definitions from ASCE 7-16. Snow and wind loads were revised to comply with ASCE 7-16. The basic wind speed map was deleted, and a reference to the basic wind speed map in ASCE 7-16 was added. Material class 0 and related unit stresses were deleted. Method 1 for determining the allowable local buckling compressive stress, F_L, for shells was revised to use the requirements of reference 11 in Sec. A.1.4, Method 2 was deleted, and Method 3 was renamed Method 2. Ladder, handrail and guardrail loads were removed, and references to OSHA and ASCE 7 were added. Additional rules for shell girders and roof knuckles were added. Design requirements for anchorages were clarified.

4. Sec. 12 was revised to update the safety factors for deep and shallow foundations based on the level of testing employed.

5. Sec. 13 was revised so that seismic load complies with ASCE 7-16. The site classification procedure was deleted, and a reference to the procedure in ASCE 7-16 was added. Spectral response acceleration maps and the long-period transition period map were deleted, and references to the maps in ASCE 7-16 were added. The vertical design acceleration was revised to include values for responses limited by buckling and responses not limited by buckling. Minimum freeboard requirements were made mandatory of all styles of tanks. The sloshing wave height equation was revised to match ASCE 7-16. The equation for allowable lateral shear for flat-bottom tanks was revised to show a service-level allowable similar to API 650 and ASCE 7-16.

6. Sec. 14 was revised to eliminate references to withdrawn material specifications (i.e., ASTM A442 and A678).

V. Comments. If you have any comments or questions about this standard, please call AWWA Engineering and Technical Services group 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.

ANSI/AWWA D100-21
(Revision of ANSI/AWWA D100-11)

American Water Works Association
Dedicated to the World's Most Important Resource®

AWWA Standard

Welded 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 welded carbon steel tanks for the storage of water at atmospheric pressure.

1.1.1 *Tank roofs.* All tanks storing potable water shall have roofs. Tanks storing nonpotable water may be constructed without roofs.

1.1.2 *Items not covered.* This standard does not cover all details of design and construction because of the large variety of sizes and shapes of tanks. Details that are not addressed shall be designed and constructed to be adequate and as safe as those that would otherwise be provided under this standard. This standard does not address composite elevated tanks (i.e., steel tanks with a concrete support structure) (see ANSI/AWWA D107, Composite Elevated Tanks for Water Storage). With the exception of aluminum dome roofs, this standard does not cover tanks constructed with materials other than carbon steel. This standard does not cover painting and disinfecting of tanks (see ANSI/AWWA D102, Coating Steel Water-Storage Tanks, and ANSI/AWWA C652, Disinfection of Water-Storage Facilities).

1.1.3 *Design method.* With the exception of reinforced concrete foundations, this standard is based on the allowable-stress design method.

Sec. 1.2 Definitions

The following definitions shall apply in this standard:

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

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

3. *Elevated tank:* A container or storage tank supported on a tower.

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

5. *Potable water:* Water that is safe and satisfactory for drinking and cooking.

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

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

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

9. *Tank:* An elevated tank, a standpipe, or a reservoir.

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

10.1 Bottom capacity level (BCL): The water level above which the specified capacity is provided. In a ground-supported tank (reservoir or standpipe), the BCL shall be the water level in the tank shell when the tank is emptied through the specified discharge fittings unless otherwise specified.

10.2 Maximum operating level (MOL): The specified maximum water level under normal operating conditions. The MOL shall be taken as the MWL unless otherwise specified.

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

Sec. 1.3 Drawings to Be Provided

Construction drawings for the foundation, tank, and accessories shall be provided. Where foundation and tank design are performed by separate parties, each party shall provide construction drawings. If anchorage is required, anchorage details, including required embedment, local reinforcement, and minimum required concrete strength, shall be provided as part of the tank design. Tanks designed according to Method 2 of Sec. 3.4.3 shall be identified as such within the construction drawings.

Details of all welded joints shall be provided when specified. Standard weld symbols as listed in AWS A2.4, Standard Symbols for Welding, Brazing, and Nondestructive Examination, shall be used, unless joint details are shown.

Sec. 1.4 References

This standard references the following documents. In the editions specified, these documents 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.

ACI* 301-10—Specifications for Structural Concrete.

ACI 318-11—Building Code Requirements for Structural Concrete and Commentary.

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

ANSI/AISC† 360-16—Specification for Structural Steel Buildings.

ANSI‡/AWWA C652-19—Disinfection of Water-Storage Facilities.

ANSI/AWWA D102-21—Coating Steel Water-Storage Tanks.

ANSI/AWWA D107-16—Composite Elevated Tanks for Water Storage.

ANSI/AWWA D108-19—Aluminum Dome Roofs for Water Storage Facilities.

API§ 5L—Specification for Line Pipe, 45th Edition.

API 620—Design and Construction of Large, Welded, Low-Pressure Storage Tanks, 12th Edition, with Addendum 1 (2014), Addendum 2 (2018), and Addendum 3 (2021).

API 650—Welded Tanks for Oil Storage, 13th Edition, with January 2021 Errata.

ASCE¶ 7-16—Minimum Design Loads and Associated Criteria for Buildings and Other Structures.

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

ASME BPVC Sec. V—Nondestructive Examination, 2013 Edition.

* American Concrete Institute, 38800 Country Club Dr., Farmington Hills, MI 48331.
† American Institute of Steel Construction, 1 East Wacker Drive, Suite 700, Chicago, IL 60601.
‡ American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036.
§ American Petroleum Institute, 1220 L Street NW, Washington, DC 20005.
¶ American Society of Civil Engineers, 1801 Alexander Bell Drive, Reston, VA 20191.
** The American Society of Mechanical Engineers, Two Park Avenue, New York, NY 10016.

ASME BPVC Sec. VIII, Div. 1—Rules for Construction of Pressure Vessels Division 1, 2013 Edition.

ASME BPVC Sec. VIII, Div. 2—Rules for Construction of Pressure Vessels Division 2, 2013 Edition.

ASME BPVC Sec. IX—Welding, Brazing, and Fusing Qualifications, 2013 Edition.

ASNT[††] SNT-TC-1A-11—Recommended Practice for Personnel Qualification and Certification in Nondestructive Testing.

ASTM[‡‡] A6/A6M-13a—Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling.

ASTM A20/A20M-13—Standard Specification for General Requirements for Steel Plates for Pressure Vessels.

ASTM A27/A27M-13—Standard Specification for Steel Castings, Carbon, for General Application.

ASTM A36/A36M-12—Standard Specification for Carbon Structural Steel.

ASTM A53/A53M-12—Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless.

ASTM A105/A105M-13—Standard Specification for Carbon Steel Forgings for Piping Applications.

ASTM A106/A106M-13—Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service.

ASTM A108-13—Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished.

ASTM A131/A131M-13—Standard Specification for Structural Steel for Ships.

ASTM A181/A181M-13—Standard Specification for Carbon Steel Forgings, for General-Purpose Piping.

ASTM A193/A193M-12b—Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications.

ASTM A283/A283M-13—Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates.

ASTM A307-14e1—Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60,000 PSI Tensile Strength.

[††] American Society for Nondestructive Testing, P.O. Box 28518, 1711 Arlingate Lane, Columbus, OH 43228.

[‡‡] ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428.

ASTM A333/A333M-13—Standard Specification for Seamless and Welded Steel Pipe for Low-Temperature Service and Other Applications with Required Notch Toughness.

ASTM A350/A350M-13—Standard Specification for Carbon and Low-Alloy Steel Forgings, Requiring Notch Toughness Testing for Piping Components.

ASTM A370-13—Standard Test Methods and Definitions for Mechanical Testing of Steel Products.

ASTM A435/A435M-90(2012)—Standard Specification for Straight-Beam Ultrasonic Examination of Steel Plates.

ASTM A500/A500M-13—Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes.

ASTM A501/A501M-14—Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing.

ASTM A516/A516M-10—Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service.

ASTM A517/A517M-10—Standard Specification for Pressure Vessel Plates, Alloy Steel, High-Strength, Quenched and Tempered.

ASTM A524-96(2012)—Standard Specification for Seamless Carbon Steel Pipe for Atmospheric and Lower Temperatures.

ASTM A537/A537M-13—Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel.

ASTM A568/A568M-13ae1—Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for.

ASTM A573/A573M-13—Standard Specification for Structural Carbon Steel Plates of Improved Toughness.

ASTM A588/A588M-10—Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi (345 MPa) Minimum Yield Point, with Atmospheric Corrosion Resistance.

ASTM A592/A592M-10—Standard Specification for High-Strength Quenched and Tempered Low-Alloy Steel Forged Parts for Pressure Vessels.

ASTM A633/A633M-13—Standard Specification for Normalized High-Strength Low-Alloy Structural Steel Plates.

ASTM A662/A662M-12—Standard Specification for Pressure Vessel Plates, Carbon-Manganese-Silicon Steel, for Moderate and Lower Temperature Service.

ASTM A668/A668M-13e1—Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use.

ASTM A992/A992M-11—Standard Specification for Structural Steel Shapes.

ASTM A1011/A1011M-14—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.

ASTM D1143/D1143M-07(2013)e1—Standard Test Methods for Deep Foundations Under Static Axial Compressive Load.

ASTM D1751-04(2013)e1—Standard Specification for Preformed Expansion Joint Filler for Concrete Paving and Structural Construction (Nonextruding and Resilient Bituminous Types).

ASTM D2026/D2026M-15—Standard Specification for Cutback Asphalt (Slow-Curing Type).

ASTM D2027/D2027M-13—Standard Specification for Cutback Asphalt (Medium-Curing Type).

ASTM D4945-17—Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.

ASTM F1554-07ae1—Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength.

ASTM F3125/F3125M— Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, Inch Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 MPa and 1040 MPa Minimum Tensile Strength.

AWS[§§] A2.4-12—Standard Symbols for Welding, Brazing, and Nondestructive Examination.

AWS A3.0M/A3.0-10—Standard Welding Terms and Definitions Including Terms for Adhesive Bonding, Brazing, Soldering, Thermal Cutting, and Thermal Spraying.

AWS A5.1/A5.1M-12—Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding.

AWS A5.5/A5.5M-06—Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding.

AWS B2.1/B2.1M-14—Specification for Welding Procedure and Performance Qualification.

[§§] American Welding Society, 8669 NW 36 St., #130, Miami, FL 33166.

AWS D1.1/D1.1M-10—Structural Welding Code—Steel.

AWS QC1-07—Standard for AWS Certification of Welding Inspectors.

CSA[¶¶] G40.21-13—Structural Quality Steel.

IBC***-12—International Building Code

OSHA[†††]—Occupational Safety and Health Standards, 29 CFR, Part 1910, latest edition.

AISI[‡‡‡] T-192—Steel Plate Engineering Data, Volumes 1 and 2, 2011 Edition.

SECTION 2: MATERIALS

Sec. 2.1 General

2.1.1 *Materials*. All materials to be incorporated in any structure to meet this standard shall be new, be previously unused, and comply with all of the requirements of this standard. Copies of the mill test reports shall be furnished when specified.

2.1.2 *Unidentified materials*. 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. When such unidentified materials are used, a report showing the test results shall be provided.

Sec. 2.2 Material Specifications

2.2.1 *Bolts, anchor bolts, and rods.* Bolts shall conform to ASTM A307, grade B, or ASTM F3125/F3125M, grade A325. Anchor bolts shall conform to ASTM A36/A36M; ASTM A193/A193M, grade B7; or ASTM F1554, grade 36, 55 (weldable), or 105. Rods shall conform to ASTM A36/A36M.

ASTM A193/A193M, grade B7 bolts and ASTM F1554, grade 105 bolts shall not be used for anchor bolts unless mild steel anchor bolts (ASTM A36/A36M or ASTM F1554, grades 36 or 55) exceed 2½ in. (63 mm) in diameter. When ASTM F1554, grade 55 anchor bolts are used, they shall comply with the weldable steel requirements of ASTM F1554, Supplement S1.

[¶¶] Canadian Standards Association, 5060 Spectrum Way, Mississauga, ON L4W 5N6.

*** International Code Council, 500 New Jersey Ave., NW, Washington, DC 20001.

[†††] Occupational Safety and Health Administration, 200 Constitution Ave. N.W., Washington, DC 20210.

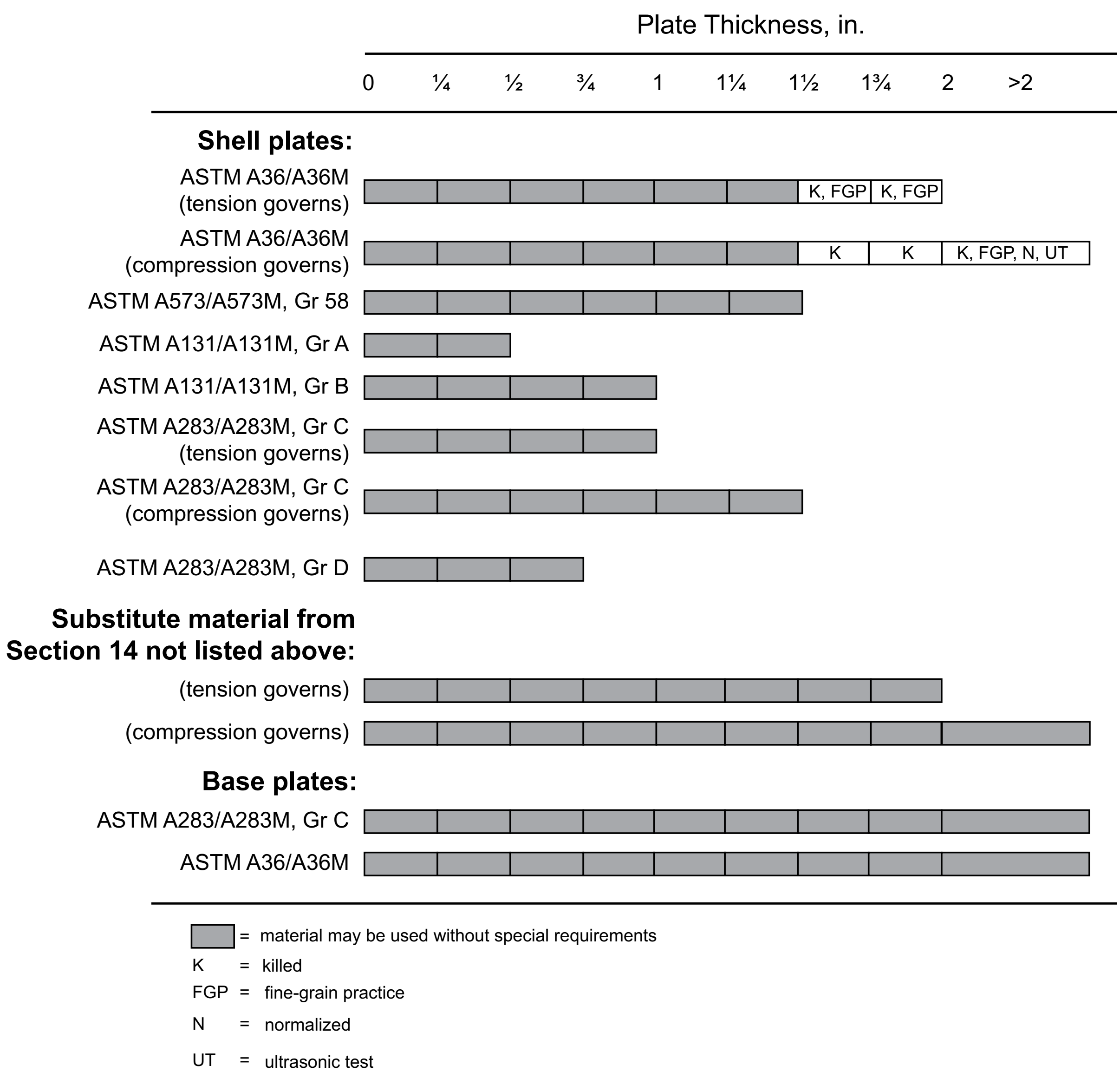
[‡‡‡] American Iron and Steel Institute, 25 Massachusetts Ave. NW, Suite 800, Washington, DC 20001.

2.2.2 *Reinforcing steel.* Reinforcing steel shall comply with the requirements of ACI 318.

2.2.3 *Plates.* Plate materials shall conform to any of the following ASTM standards: A36/A36M; A131/A131M, grades A and B; A283/A283M, grades C and D; or A573/A573M, grade 58; and materials listed in Sec. 2.2.3.2 and Sec. 2.2.7.

2.2.3.1 Thickness limitations and special requirements. Plate thickness limitations and special requirements shall be as provided in the following subsections and summarized in Table 1.

Table 1 Thickness limitations and special requirements



2.2.3.1.1 ASTM A36/A36M shell plates governed by tension stress shall be limited to a thickness of 2 in. (51 mm), and the material shall be killed and manufactured to a fine-grain practice for thicknesses greater than 1½ in. (38 mm).

When compression governs, ASTM A36/A36M shell plates greater than 1½ in. (38 mm) and less than or equal to 2 in. (51 mm) in thickness shall be killed. Plates in compression, such as compression rings (biaxial compression), parts of the primary support system, and the primary container shell, may not exceed 2 in. (51 mm) in thickness unless the material is killed, manufactured to a fine-grain practice, normalized, and ultrasonically inspected to the acceptance criteria of ASTM A435/A435M.

2.2.3.1.2 ASTM A131/A131M, grade A shall not be used in thicknesses greater than ½ in. (13 mm). ASTM A131/A131M, grade B shall not be used in thicknesses greater than 1 in. (25 mm).

2.2.3.1.3 ASTM A283/A283M, grade C shell plates are limited to a thickness of 1 in. (25 mm) when tension stress governs and 1½ in. (38 mm) when compression stress governs. ASTM A283/A283M, grade D shell plates are limited to a thickness of ¾ in. (19 mm).

2.2.3.1.4 ASTM A573/A573M, grade 58 plates are limited to 1½ in. (38 mm) in thickness.

2.2.3.1.5 Where details are such that tension may occur through the plate thickness, consideration shall be given to the possibility that lamellar tearing may occur.

2.2.3.1.6 ASTM A36/A36M or ASTM A283/A283M, grade C, steels may be used for base plates regardless of thickness or temperature. ASTM A36/A36M steel ordered as a bearing plate in accordance with ASTM A36/A36M, Sec. 5.2, is not acceptable.

2.2.3.2 Substitute material. When material supply or shortages require the use of substitute materials, Category 1 and 2 materials from Sec. 14 may be used for tanks designed in accordance with Sec. 3, without regard to the thickness and temperature limitations of Sec. 14. Stress levels for substitute material shall be limited to those in Sec. 3.

2.2.3.3 Basis of providing plates. Plates may be provided on the weight basis with permissible underrun and overrun, according to the tolerance table for plates ordered to weight published in ASTM A6/A6M.

2.2.4 *Sheets*. Sheet materials shall conform to ASTM A1011/A1011M, grade 30, 33, or 36, or ASTM A568/A568M. Sheet materials may only be used for roofs, platforms, and nonstructural items.

2.2.5 *Structural shapes*. All structural shapes for use under the provisions of this standard shall be produced by the open-hearth, basic-oxygen, or electric-furnace process.

2.2.5.1 Nontubular. Open or nontubular structural shapes shall conform to ASTM A36/A36M or ASTM A992/A992M. When structural shapes are fabricated from plates, the plate materials shall conform to Sec. 2.2.3 of this standard.

2.2.5.2 Tubular. Tubular structural shapes may be used for structural components, such as columns, struts, and miscellaneous parts. Such tubular shapes may be circular, square, rectangular, or other cross sections. Structural tubing with square or rectangular cross sections shall comply with one of the following specifications:

1. Cold-formed structural tubing shall comply with ASTM A500/A500M.
2. Hot-formed tubing shall comply with ASTM A501/A501M.

2.2.5.2.1 Structural tubing with other cross sections may be manufactured from plates of any of the specifications permitted in Sec. 2.2.3, provided that the welding and other manufacturing processes are in compliance with all sections of this standard.

2.2.5.2.2 Steel pipe may be used as tubular structural members, provided that it complies with ASTM A106/A106M, grade B; ASTM A53/A53M type E or S, grade B; or API 5L, grade B.

2.2.6 *Pins*. Pins shall comply with ASTM A307, grade B; ASTM A108, grade 1018 or 1025, conforming to supplemental requirement S9* to meet a minimum yield strength of 30,000 psi; or ASTM A36/A36M. Size and diameter tolerances on turned pins shall be equal to those of cold-finished shafting. Surface finish shall depend on application, but in no case shall the surface finish be rougher than 125 μin. (3.175 μm).

2.2.7 *Canadian steels*. Canadian steels acceptable for use under this standard are CSA G40.21, grades 38W, 38WT, 44W, and 44WT. All four grades of CSA G40.21 will have allowable design stresses per class 2 (see Sec. 3).

2.2.8 *Cast steel*. Castings shall conform to ASTM A27/A27M, grade 60-30 (full annealed).

2.2.9 *Forgings.*

2.2.9.1 Forgings shall conform to any of the following ASTM specifications: A668/A668M, class D; A181/A181M, grade II; or A105/A105M.

* S9 is needed to ensure adequate strength because ASTM A108 has no specified yield or ultimate strength.

2.2.9.2 Pipe flanges. Forged and rolled pipe flanges shall conform to the material requirements for forged carbon-steel flanges as specified in ASME B16.5.

2.2.10 *Filler metals and fluxes*. The filler metals and fluxes shall be of the same classification as those that have been qualified for each welding procedure, in accordance with Sec. 8.2.

2.2.11 *Pipe for fluid conductors*. Inlet, outlet, overflow, and other pipes, and all fittings for fluid use shall be specified.

Steel pipe shall conform to ASTM A53/A53M, type E or S, grade B; ASTM A106/A106M; or API 5L or equal. Unless otherwise specified, joints may be screwed, flanged, or welded. Other pipe materials may be specified, provided that they conform to recognized national or industry standards.

SECTION 3: GENERAL DESIGN

Sec. 3.1 Design Loads

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

3.1.1 *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 into one of the following risk categories, and the risk category shall be specified. Risk Category IV shall be used if none is specified. Risk Category I shall not be used.

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

3.1.1.2 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.

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

3.1.2 *Dead load*. Dead load shall be the estimated weight of permanent construction. 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.

3.1.3 *Water load*. Water load shall be the weight of all of the water when the tank is filled to the MWL. The unit weight used for water shall be 62.4 lb/ft^3 (1,000 kg/m^3). The weight of water in a wet riser, which is supported directly on a foundation, shall not be considered a vertical load on the riser.

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

3.1.4.1 Ground snow load. Ground snow load, p_g, shall be in accordance with ASCE 7, Figure 7.2-1 and Table 7.2-1. Ground snow load shall be specified for areas designated CS and for sites at elevations above the limits indicated in ASCE 7, Figure 7.2-1. 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.

3.1.4.2 Flat roof snow load. Flat roof snow load, p_f, shall be determined using the equation:

$$p_f = 0.7C_eC_tI_sp_g \tag{3-1}$$

(3-1)*

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-air 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

3.1.4.3 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°. For portions of the roof with a roof angle less than 70°, the sloped roof snow load shall be determined using the following equations:

$$C_s = \frac{70 - \theta_r}{55}, \text{ where } 0 \le C_s \le 1 \tag{3-2}$$

* For equivalent metric equation, see Sec. 3.14.

$$p_s = C_s p_f \tag{3-3}$$

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

3.1.4.4 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° and portions of double-curved roofs with a roof angle less than 15°. 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 $20I_s$.

3.1.4.5 Snow load on horizontal surfaces. Flat roof snow load shall be applied to circumferential external tank elements having a horizontal projection such as stiffeners, balconies, and wind girders.

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

3.1.6 *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.

3.1.6.1 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, Figures 26.5-1A-D and 26.5-2A-D, and the assigned risk category.

3.1.6.1.1 Special wind regions. The basic wind speed for mountainous terrain, gorges, and special wind regions shown in ASCE 7, Figures 26.5-1A-D, 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 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.

3.1.6.1.2 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/h); Hawaii; Puerto Rico; Guam; Virgin Islands; and American Samoa. The basic wind speed for hurricane-prone regions shall be in accordance with ASCE 7, Figures 26.5-1A-D and 26.5-2A-D, or may be determined from simulation techniques in accordance with ASCE 7.

3.1.6.2 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 is the minimum exposure category allowed and shall be used unless it is determined that Exposure D is applicable or specified.

3.1.6.2.1 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.

3.1.6.2.2 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.

3.1.6.3 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, K_{zt}, shall be 1.0 unless otherwise specified.

3.1.6.4 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.

3.1.6.5 Velocity pressure. Velocity pressure, q_z, at height above ground level, z, shall be determined using the equation:

$$q_z = \lambda_w \left[0.00256 K_z K_{zt} K_d V^2 \right] \tag{3-4}$$ *

* For equivalent metric equation, see Sec. 3.14.

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 2, in pounds per square foot (see Appendix A, Figure A.1, for examples)

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

= 0.6

K_z = velocity pressure exposure coefficient in accordance with Table 2

K_{zt} = topographic factor in accordance with Sec. 3.1.6.3

K_d = directionality factor

= 1.0

V = basic wind speed, in miles per hour

Table 2 Velocity pressure exposure coefficient, K_z

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	1.52
160	48.8	1.39	1.55
180	54.9	1.43	1.58
200	61.0	1.46	1.61
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 26.10-1.

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

$$p_w = q_z G C_f \geq 30 C_f \tag{3-5}$$ (3-5)*

Where:

p_w = wind design pressure, in pounds per square foot
G = gust-effect factor in accordance with Sec. 3.1.6.4
C_f = wind force coefficient in accordance with Sec. 3.1.6.6.1

The other symbol has been previously defined in this section.

3.1.6.6.1 Wind force coefficient. 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 3. Wind force coefficients for shapes and conditions not addressed in Table 3 shall be in accordance with ASCE 7.

Table 3 Wind 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° and $\left(d\sqrt{q_z} > 2.5\right)$	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° and $\left(d\sqrt{q_z} \leq 2.5\right)$	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 pilasters or fluted section, 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. 3.14.

3.1.6.7 Wind load, *W*. Wind load, *W*, shall be determined by applying the design wind pressures, p_w, to the projected areas, A_f, of the tank and support structure. Wind loads shall be applied at the centroids of the projected areas within each height zone defined in Table 2.

3.1.6.7.1 Wind load on columns, struts, and sway rods. For columns and struts of structural shapes, the projected areas shall be calculated. It shall be assumed that struts on the leeward side of the tower are shielded 50 percent by those on the windward side. In the case of columns and sway rods, the design wind pressure shall be applied to the projected area of each member. The wind load in any direction on structural columns, other than tubular columns, shall be based on the larger of the two projected areas—one on the vertical plane containing the longitudinal axis of the column and the vertical axis of the tank and support structure, and the other on a vertical plane perpendicular to the first.

3.1.6.7.2 Shrouds. Where structures may be totally enclosed in a shroud during painting, the structure shall be checked for a wind velocity 50 percent greater than the maximum operating wind velocity of the shroud, but not more than 50 mph (22 m/s) if the maximum operating wind velocity is unknown. The projected area of the shroud shall be the same height as the shroud or 6 ft (1.6 m) minimum above the maximum height of the structure and 6 ft (1.6 m) wider than the projected area of the structure, unless otherwise specified.

3.1.6.7.3 Wind–structure interaction. The effects of wind–structure interaction, such as vortex shedding, shall be considered for slender single-pedestal tanks and slender standpipes.

3.1.7 *Seismic load*. Structures shall be designed for seismic loads as defined in Sec. 13. See Sec. 3.1.7.1 for exception.

3.1.7.1 Seismic load exception. Structures located where the mapped spectral response acceleration at 1-s period, S_1, is less than or equal to 0.04*g* and the mapped short-period spectral response acceleration, S_S, is less than or equal to 0.15*g* do not require design for seismic loads. See Sec. 13.2 for definitions.

3.1.7.2 Bracing details. For multicolumn tanks located where design for seismic loads is not required (Sec. 3.1.7.1), bracing that is part of the lateral force-resisting system shall be detailed to provide ductility in the event of an overload condition.

3.1.7.3 Elevated tanks. For elevated tanks, design horizontal forces are calculated by taking the total of dead weight plus water weight and multiplying by the appropriate design acceleration. The forces are assumed to act through the center of gravity of the masses that cause them.

3.1.7.4 Flat-bottom tanks. For flat-bottom tanks resting on the ground, design horizontal forces are calculated by multiplying the dead weight of the tank and the effective mass of the water by the appropriate design acceleration. The horizontal force caused by dead weight is assumed to act through the center of gravity. The effective mass of the water and the heights at which the resulting design horizontal forces are assumed to act shall be calculated using Sec. 13.5.2.2.

3.1.8 *Balcony, platform, ladder, stair, and localized roof loads.* A vertical load (and only one such load in each case) shall be applied as follows: 1,000 lb (454 kg) to any 10-ft^2 (0.93-m^2) area on the balcony floor, 1,000 lb (454 kg) to each platform, and 500 lb (227 kg) to any 10-ft^2 (0.93-m^2) area on the tank roof. Distributed and concentrated live loads for ladders and stair systems shall be in accordance with ASCE 7 and OSHA regulations. Structural parts and connections shall be properly proportioned to withstand such loads. The previously mentioned loads need not be combined with the design snow load specified in Sec. 3.1.4 but shall be included in all other applicable load combinations. The balcony, platform, and roof plating may deflect between structural supports to support the loading. Any additional design considerations must be specified. See Commentary to this standard, Appendix A, Sec. A.3.1.8, for possible additional considerations.

3.1.9 *Handrail and guardrail assemblies.* Distributed and concentrated live loads for handrail and guardrail assemblies shall be determined in accordance with ASCE 7 and OSHA regulations.

Sec. 3.2 Unit Stresses

Except for roof supports, stress combinations specified in Sec. 3.3.3, and other exceptions specifically provided for elsewhere in this standard, all steel members shall be so designed and proportioned that, during the application of any of the loads previously specified, or any required combination of these loads, the maximum stresses shall not exceed those specified in Tables 5 through 9. Based on their published minimum yield strength, F_y, materials are divided into two classes for determining the allowable design stress (see Table 4). Allowable unit stress values, wherever stated in this standard for tank plate joints, shall be reduced by the applicable joint efficiencies stated in Table 11.

Table 4 Material classes

	F_y*	
Class	***psi***	**(*MPa*)**
1	$30,000 \le F_y \le 34,000$	$(206.8 \le F_y \le 234.4)$
2	$F_y > 34,000$	$(F_y > 234.4)$

* Where F_y is the published minimum yield strength.

Table 5 Unit stresses—tension

		Maximum Unit Stress	
Item	**Class**	***psi***	**(*MPa*)**
Plates in tank shell	1, 2*	15,000	(103.4)
Structural steel, built-up structural members, structural details	1 2	15,000 18,000	(103.4) (124.1)
Tension rings	1, 2	15,000	(103.4)
Bolts and other nonupset threaded parts†		15,000	(103.4)
Anchor bolts†			
Mild steel‡			
ASTM A36/A36M or ASTM F1554-36		15,000	(103.4)
ASTM F1554-55 (weldable)		18,750	(129.3)
High-strength steel			
ASTM A193/A193M, Gr B7			
1¼ in. ≤ diameter ≤ 2½ in.		31,250	(215.5)
2½ in. < diameter ≤ 4 in.		28,750	(198.3)
ASTM F1554-105			
1¼ in. ≤ diameter ≤ 3 in.		31,250	(215.5)
Bracing rods with swedged (upset) or welded, enlarged stub ends§	1 2	15,000 18,000	(103.4) (124.1)
Cast steel		11,250	(77.6)

* See exceptions in Sec. 4.5.
† On area based on diameter at root of threads.
‡ See Sec. 3.3.3.2 for additional requirements.
§ On area based on diameter at root of threads or an area of plain portion of rod, whichever is smaller.

Table 6 Unit stresses—compression

Item	Class	Maximum Unit Stress *psi*	*(MPa)*
Plates in tank shell, structural steel, built-up members, plate in structural applications, structural details, and weld metal	1 2	15,000 18,000	(103.4) (124.1)
Columns, struts, and double-curved, conical, and cylindrical shell plates		*	
Plate girder stiffeners		15,000	(103.4)
Webs of rolled sections at toe of fillet		18,000	(124.1)
Compression rings		15,000	(103.4)
Cast steel		15,000	(103.4)

* See Sec. 3.4 for stability requirements.

Table 7 Unit stresses—primary bending

Item	Class	Maximum Unit Stress *psi*	*(MPa)*
Tension on extreme fibers of rolled sections, built-up members, and plate girders, except column base plates, roof plates, roof rafters, roof stiffeners, and roof supports	1 2	15,000 18,000	(103.4) (124.1)
Compression on extreme fibers of rolled sections, built-up members, and plate girders, except column base plates, roof plates, roof rafters, roof stiffeners, and roof supports:			
$LD/BT^* \leq 600$	1	15,000	(103.4)
$LD/BT^* > 600$	1	9,000,000/(*LD*/*BT*)	(62,040/[*LD*/*BT*])
$LD/BT^* \leq 500$	2	18,000	(124.1)
$LD/BT^* > 500$	2	9,000,000/(*LD*/*BT*)	(62,040/[*LD*/*BT*])
Column base plates, roof plates	1 2	20,000 24,000	(137.9) (165.4)
Compression on extreme fibers of double-curved, conical, or cylindrical shell plates (full section stress, not through thickness bending)		See Sec. 3.4.	
Pins, extreme fiber		22,500	(155.1)
Cast steel		11,250	(77.6)

* Where *L* is the laterally unsupported length and *D* is the depth of the member, *B* is the width, and *T* is the thickness of its compression flange, all in inches.

Table 8 Unit stresses—shearing

Item	Class	Maximum Unit Stress *psi*	*(MPa)*
Plates in tank shell, structural connections, structural details; also, webs of beams and plate girders, gross section	1 2	12,000 14,400	(82.7) (99.3)
Pins and turned bolts in reamed or drilled holes		11,250	(77.6)
Unfinished bolts		7,500	(51.7)
Cast steel		7,325	(50.5)

Table 9 Unit stresses—bearing

Item		Maximum Unit Stress *psi*	*(MPa)*
Pins and turned bolts in reamed or drilled holes		24,000	(165.5)
Contact area of milled surfaces		22,500	(155.1)
Contact area of fitted stiffeners		20,250	(139.6)
Expansion rollers and rockers		600*d**	
Concrete bearing shall conform to ACI 318			
Machined finished bolts in reamed or drilled holes	Double shear Single shear	30,000 24,000	(206.8) (165.5)
Unfinished bolts	Double shear Single shear	18,750 15,000	(129.3) (103.4)

* In which *d* is the diameter of roller or rocker, in inches, and unit stress is in pounds per linear inch of roller or rocker.

3.2.1 *Width-to-thickness limitations.* The ratio of width to thickness of elements subject to axial compression or compression caused by bending, or both, other than those addressed in Sec. 3.4.2, Sec. 3.4.4, and Sec. 3.5 shall not exceed the limits shown for noncompact sections in ANSI/AISC 360, Table B4.1.

3.2.2 *Pipe thickness underrun.* Potential thickness underrun as permitted by the selected steel pipe specification shall be considered in calculating actual and allowable stresses in tubular structural members.

3.2.3 *Plate thickness underrun.* Plate thickness underrun less than or equal to 0.01 in. (0.3 mm) is permitted without adjusting stresses.

Sec. 3.3 Combined Stresses

3.3.1 *Axial and bending stresses.* Unless specifically provided for elsewhere in this standard, members subject to both axial and bending stresses shall be proportioned in accordance with Eq 3-6:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \tag{3-6}$$

Where:

F_a = the axial unit stress that would be permitted by this standard if axial stress only existed

F_b = the bending unit stress that would be permitted by this standard if bending stress only existed

f_a = the axial unit stress (actual), equal to axial load divided by cross-sectional area of member

f_b = the bending unit stress (actual), equal to bending moment divided by section modulus of member

Refer to Sec. 3.6 for requirements for roof supports.

3.3.2 *Bolts*. Bolts that are subject to shearing and tensile forces shall be so proportioned that the combined unit stress will not exceed the allowable unit stress for bolts in tension only. Bolts in tension shall have heads shaped to provide adequate shearing strength through the heads.

3.3.3 *Seismic, wind, and other forces*. Members subject to stresses produced by wind or seismic loads may be proportioned for unit stresses one-third greater than those specified in Sec. 3.2 and Sec. 3.4, but in no case shall the selected section be less than that required for the combination of dead and live loads specified in Sec. 3.1.2, Sec. 3.1.3, Sec. 3.1.4, and Sec. 3.1.5, using the unit stresses given in Sec. 3.2 and Sec. 3.4. Snow load need not be included with wind or seismic loads, unless otherwise specified.

3.3.3.1 Wind and seismic stresses. It is not necessary to combine wind and seismic stresses, providing each member is proportioned for the larger effect when combined with other forces.

3.3.3.2 Allowable anchor-bolt stress for seismic loads. The maximum tensile stress on the minimum root area for mild steel anchors designed for seismic loads defined in Sec. 13 shall be the lesser of 80 percent of the minimum published yield stress or 50 percent of the minimum published ultimate tensile stress.

3.3.4 *Struts*. Struts designed to resist bracing loads shall be designed as beam-columns. Bending shall include the effects of strut dead load and eccentricity caused by dead-load deflection. For seismic designs, struts shall also be checked by ultimate strength design (load factor = 1.0) to resist yield stress loads in bracing.

Sec. 3.4 Allowable Compressive Stresses for Columns, Struts, and Shells

3.4.1 *General.* This section applies to columns, struts, and shells subject to compressive loads from static, wind, or seismic load cases. Method 1 shall be used to determine the allowable local buckling compressive stress for ground-supported flat-bottom tank shells.

3.4.1.1 Notation. Notation used in Sec. 3.4.1 through Sec. 3.4.3 and Eqs 3-7 through 3-30 is defined as follows:

A = buckling coefficient

B = buckling coefficient

C_c = column slenderness ratio at which overall elastic column buckling will begin

C_o = elastic buckling coefficient

C_p = elastic buckling coefficient for pressure stabilization

D_1 = buckling coefficient

D_2 = buckling coefficient

E = modulus of elasticity of shell material, in pounds per square inch

= 29,000,000 psi (200,000 MPa)

F_a = allowable axial compressive stress, including local buckling and slenderness effects, in pounds per square inch

F_b = allowable bending compressive stress, including local buckling effects, in pounds per square inch

F_{ch} = compressive failure stress in the presence of circumferential tension at the point of consideration, in pounds per square inch

F_{cr} = critical buckling stress, in pounds per square inch

F_{eff} = effective stress, in pounds per square inch

F_L = allowable local buckling compressive stress, in pounds per square inch

F_y = minimum published yield strength, in pounds per square inch

f_a = calculated stress in member because of axial load, in pounds per square inch

f_b = calculated stress in member because of bending moment, in pounds per square inch

f_h = circumferential membrane tension stress because of hydrostatic pressure, in pounds per square inch

K = effective column length factor (Commentary of ANSI/AISC 360)
= 1.0 for pinned end columns or struts
= 2.0 for cantilever columns, such as the shaft of a single-pedestal tank

K_φ = slenderness reduction factor

p = hydrostatic pressure, in pounds per square inch

R = radius of exterior surface of the shell, normal to the plate at the point under consideration and measured from the exterior surface of the plate to the axis of revolution, in inches

r = radius of gyration of the section, in inches

L = member length, in inches

t = thickness of the shell plate, in inches

t_{base} = required thickness of shell plate based on the allowable local buckling compressive stress, F_L, by Method 1 and a minimum specified yield strength, F_y, of 36,000 psi (248.2 MPa)

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

η = plasticity reduction factor

Δ = buckling coefficient

3.4.1.2 Axial load. The allowable compressive stress due to axial load is given by:

$$F_a = F_L K_\varphi \tag{3-7}$$

3.4.1.3 Bending moment. The allowable compressive stress due to bending moment is given by:

$$F_b = F_L \tag{3-8}$$

3.4.1.4 Axial load and bending moment. The combined effects of axial load and bending moment interaction must satisfy the following:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{f_a}{K_\varphi F_L} + \frac{f_b}{F_L} \le 1.0 \tag{3-9}$$

3.4.1.5 Local buckling. The effects of local buckling shall be considered. See Sec. 3.4.2 for structural sections and Sec. 3.4.3 for double-curved axisymmetrical, conical, and cylindrical shell sections.

3.4.1.6 Slenderness. The slenderness reduction factor, K_φ, shall be calculated as shown in the following:

When $25 < \frac{KL}{r} \le C_c'$, $$K_\varphi = 1 - \frac{1}{2}\left(\frac{\frac{KL}{r}}{C_c'}\right)^2 \tag{3-10}$$

When $\frac{KL}{r} \ge C_c'$, $$K_\varphi = \frac{1}{2}\left(\frac{C_c'}{\frac{KL}{r}}\right)^2 \tag{3-11}$$

When $\frac{KL}{r} \le 25$, $$K_\varphi = 1.0 \tag{3-12}$$

$$C_c' = \sqrt{\frac{\pi^2 E}{F_L}} \tag{3-13}$$

3.4.2 *Structural sections.* The maximum permissible unit stress in compression for built-up and structural columns or struts shall be determined from Eqs 3-7, 3-8, and 3-10 through 3-13. The value of F_L is the following:

F_L for class 1 materials = 15,000 psi (103.41 MPa)

F_L for class 2 materials = 18,000 psi (124.10 MPa)

The aforementioned allowable stresses shall be reduced to account for stability of stiffened and unstiffened elements in accordance with Sec. 3.4.2.1.

3.4.2.1 Stiffened and unstiffened elements. Stiffened and unstiffened elements subject to axial compression or compression caused by bending shall be considered as fully effective when the ratio of width to thickness does not exceed the limits of ANSI/AISC 360, Table B4.1. When the ratio exceeds the limit, the allowable stress shall be reduced in accordance with ANSI/AISC 360.

3.4.2.2 Slenderness ratio. Maximum permissible slenderness ratios, KL/r, shall be as follows: for compression members carrying weight or pressure of tank contents, 120; for compression members carrying loads from wind or seismic, or both, 175. See Sec. 3.6 for columns carrying roof loads only.

3.4.3 *Double-curved axisymmetrical, conical, and cylindrical sections.* Two methods are provided for calculating the allowable local buckling compressive stress, F_L—Methods 1 and 2. The requirements for Methods 1 and 2 are given in Sec. 3.4.3.1 and Sec. 3.4.3.2, respectively.

The maximum unit stress in compression caused by axial load and bending moment on the cross section is limited to the values defined in this section. This section is used when the meridional membrane stress in the shell or support containing the longitudinal axis of the structure or member is compressive and the stress normal to the compressive stress is tension or no stress at all (i.e., biaxial tension-compression or uniaxial compression). Biaxial compression, when compressive stresses are present in both directions, requires further analysis and is beyond the scope of this standard. See Sec. 10.6.6 if the allowable stresses in this section are applied. Where the tolerances of Sec. 10.6.6 are not met, further evaluation is required and corrective action, such as reworking the shell or adding stiffeners, may be required.

3.4.3.1 Method 1. Method 1 is a simplified design method that is based on membrane analysis techniques. This method is mandatory for shells and supports that do not contain water. The allowable local buckling compressive stress, F_L, shall be determined using the following equations. The minimum specified yield strength, F_y, used for design shall not exceed 36,000 psi (248.2 MPa).

$$F_L = \frac{233F_y}{2\left(166 + R/t\right)} \le \frac{F_y}{2} \qquad \text{for } R/t \le \left(R/t\right)_c \tag{3-14}$$

$$F_L = \frac{C_o E}{2\left(R/t\right)} \qquad \text{for } R/t > \left(R/t\right)_c \tag{3-15}$$

The ratio $(R/t)_c$ shall be in accordance with Table 10, and the elastic buckling coefficient, C_o, shall be determined using the following equation:

$$C_o = \frac{1022}{195 + R/t} \ge 0.125 \tag{3-16}$$

Table 10 Values of $(R/t)_c$

F_y (*psi*)	30,000	32,000	34,000	36,000	38,000	40,000
(*MPa*)	206.8	220.6	234.4	248.2	262.0	275.8
$(R/t)_c$	403	377	354	334	316	299

3.4.3.2 Method 2. Method 2 is a complex design method based on a nonlinear buckling analysis. Method 2 only applies to water-filled shells that meet the limitations of Sec. 3.4.3.2.1. Method 2 allows an increase in the allowable local buckling compressive stress because of the stabilizing effect of hydrostatic pressure.

3.4.3.2.1 Method 2 only applies to water-filled cylinders, cones, and double-curved shell elements that meet the following requirements:

1. The plate thickness, t, determined by this method shall not be less than $0.8t_{base}$ for radius-to-thickness ratios less than or equal to 700, and $0.7t_{base}$ for ratios greater than 700.
2. The water shall be contained within the shell such that hydrostatic pressure causes circumferential (hoop) tension stress in the shell.
3. The angle of the shell measured from the axis of revolution to the inside shell surface shall not exceed 60°.
4. The shell elements shall be joined by butt-welded joints with complete penetration. No lap welded joints are permitted.
5. The radius-to-thickness ratio shall be less than 1,000 but greater than or equal to the value $(R/t)_c$ given in Table 10. Method 1 (Sec. 3.4.3.1) shall be used for portions of the shell where the radius-to-thickness ratio is less than the $(R/t)_c$ value given in Table 10.
6. The material of construction shall have a minimum specified yield strength, F_y, equal to or greater than 36,000 psi (248.2 MPa).

3.4.3.2.2 A nonlinear buckling analysis meeting the requirements of Sec. 3.4.3.2.3 shall be used to determine the critical buckling stress, F_{cr}. The allowable local buckling compressive stress, F_L, is the smaller of the values determined by Eqs 3-17 and 3-18:

$$F_L = \frac{F_{cr}}{2} \tag{3-17}$$

$$F_L = \left(C_o + C_p\right)\frac{\eta E}{2\left(R/t\right)} \tag{3-18}$$

The elastic buckling coefficient, C_o, shall be determined by Eq 3-16. The elastic buckling coefficient for pressure stabilization, C_p, shall be determined by the following equations:

$$C_p = \frac{1.06}{3.24 + \dfrac{E}{f_b\left(R/t\right)}} \quad \text{for cylindrical and double-curved shells} \tag{3-19}$$

$$C_p = \frac{1.33}{3.33 + \dfrac{E}{f_b\left(R/t\right)}} \quad \text{for conical shells} \tag{3-20}$$

The plasticity reduction factor, η, shall be determined by the following iterative procedure. The plasticity reduction factor shall be based on a minimum specified yield strength, F_y, of 40,000 psi (275.9 MPa) when the material of construction has a minimum specified yield strength greater than 40,000 psi (275.9 MPa).

1. Assume a value for the compressive failure stress in the presence of circumferential tension, F_{ch}.
2. Calculate the effective stress, F_{eff}, and buckling coefficient, Δ, using the following equations:

$$F_{eff} = \sqrt{F_{ch}^2 + F_{ch} f_b + f_b^2} \tag{3-21}$$

$$\Delta = \left(C_o + C_p\right)\left(\frac{Et}{\mathrm{R}}\right)\left(\frac{F_{eff}}{F_{ch} F}\right) \tag{3-22}$$

3. Use the buckling coefficient, Δ, determined in step 2 and the following equations to calculate the plasticity reduction factor, η. F_y is in units of psi.

$$\eta = 1 \quad \text{for } \Delta \le D_1 \tag{3-23}$$

$$\eta = \frac{A}{B + \Delta^{0.80}} \quad \text{for } D_1 < \Delta < D_2 \tag{3-24}$$

$$\eta = \frac{1}{\Delta} \quad \text{for } \Delta \ge D_2 \tag{3-25}$$

$$A = 0.651 + 3.83(10)^{-6} F_y \qquad \text{(3-26)*}$$

$$B = 0.355 - 2.92(10)^{-6} F_y \qquad \text{(3-27)*}$$

$$D_1 = \frac{F_y}{136{,}000} + 0.196 \qquad \text{(3-28)*}$$

$$D_2 = \frac{118.34}{F_y^{0.0573}} - 60.17 \qquad \text{(3-29)*}$$

4. Calculate a new compressive failure stress, F_{ch}, using the plasticity reduction factor, η, determined in step 3 and the following equation:

$$F_{ch} = \left(C_o + C_p\right)\frac{\eta E}{\left(R/t\right)} \qquad \text{(3-30)}$$

5. If the compressive failure stress, F_{ch}, calculated in step 4 equals the compressive failure stress, F_{ch}, assumed in step 1, then the plasticity reduction factor, η, calculated in step 3 is valid. Otherwise, assume a new compressive failure stress, and repeat steps 2, 3, 4, and 5.

3.4.3.2.3 Requirements of nonlinear buckling analysis. The nonlinear buckling analysis shall comply with the following requirements:

1. The analysis shall be based on numerical solutions using finite-element, finite-differences, or numerical-integration techniques. The analysis shall include the effect of material and geometric nonlinearities.

2. The analysis shall consider initial imperfections and gross structural discontinuities such as shell discontinuity junctures, changes in plate thickness, and plate misalignment. The magnitude of the imperfection shall not be less than e_x (see Sec. 10.6.6). The length of the imperfection shall be equal to or less than L_x (see Sec. 10.6.6) and shall be appropriate for the type of construction. The location and shape of initial imperfections shall produce the lowest critical buckling stress, F_{cr}.

* For equivalent metric equation, see Sec. 3.14.

3. The location of boundaries and boundary conditions shall produce displacements and rotations at the boundaries similar to those of the actual structure.

4. The hydrostatic pressure shall not exceed the hydrostatic pressure at operating conditions. The incremental loads required to force instability or nonconvergence shall be added as a meridional load to the shell.

5. The material of construction shall be represented by a stress–strain curve that includes the effect of residual stresses caused by fabrication and welding. Alternatively, a stress–strain curve that does not include the effect of residual stresses may be used, provided that the magnitude of the initial imperfection is equal to or greater than $2e_x$.

6. The critical buckling stress, F_{cr}, shall be determined for each shell course of different thickness. The analysis shall be based on the thickness of each shell course less the specified corrosion allowance.

3.4.4 *Flat-plate elements used in single-pedestal tanks.* The effective design-width ratio between stiffened edges of bent-plate compression elements subjected to gravity loads shall be determined by the formula:

$$l_e = \frac{b}{t} = \frac{7{,}300}{\sqrt{f}}\left[1.0 - \frac{1{,}590}{\left(\frac{w}{t}\right)\sqrt{f}}\right] \qquad (3\text{-}31)^*$$

Where:

l_e = effective design-width ratio, $0 < l_e \leq w/t$

b = effective design-width between stiffened flanges, in inches

w/t = flat width ratio

f = actual stress in the compression element width, in pounds per square inch

t = plate thickness < 1 in. (25.4 mm)

w = flat width between stiffened edges exclusive of radii, in inches

NOTE: The value of w shall not be less than the work point width less $6t$.

When compression elements are designed for gravity plus wind or seismic loads, the effective width shall be calculated for a stress equal to 0.75 times the stress caused by wind or seismic loads plus gravity loads.

* For equivalent metric equation, see Sec. 3.14.

Flat-plate elements other than those addressed in this section shall be designed using the allowable stress design provisions of ANSI/AISC 360.

Sec. 3.5 Top Shell Girder, Intermediate Shell Girders, Tension Rings, and Compression Rings

3.5.1 *General requirements for top shell girders and intermediate shell girders.* Design and details of top shell girders and intermediate shell girders shall comply with the following general requirements in addition to the specific requirements of this section. Where the terms "stiffener ring," "stiffener components," and "stiffener component" are used herein, they shall refer to all the components of top shell girders and intermediate shell girders.

3.5.1.1 Weld of girder to shell. Welds joining stiffener components to the tank shall comply with the requirements of the structural design properties of the composite section. Where structural welds are not required, all points of interface between the stiffener components and the tank and all points of interface between individual stiffener components shall comply with the requirements of Sec. 8.16.

3.5.1.2 Girder splice welds. If full composite section properties are used in the design, complete joint penetration welds shall be made at all splice locations. Where complete joint penetration welds are not used, that portion of the composite section not joined with a complete joint penetration weld shall not be included when calculating the composite section properties.

3.5.1.3 Bracing. Out-of-plane bracing of the stiffener components shall be provided as required by this section.

3.5.1.3.1 Girder bracing. Where the dimension of the stiffener component's horizontal leg or web exceeds 16 times the leg or web thickness, supports or braces shall be provided to the outstanding flange. The supports shall be spaced at the intervals required for the dead load plus roof live load or dead load plus balcony load if girder is used as a walkway. Support spacing shall not exceed 24 times the width of the outstanding flange.

3.5.1.3.2 Alternate bracing. In lieu of the above requirements, the spacing of supports and bracing for the horizontal leg or web and outstanding flange, if any, may be determined in accordance with the requirements for lateral bracing using the allowable stress design provisions of ANSI/AISC 360.

3.5.1.4 Drain holes. Stiffener components that may trap liquid shall be provided with adequate drain holes. Uninsulated tanks having a stiffener ring shall have small water-shedding slopes and/or drain holes or slots unless the purchaser approves an alternate means of drainage. If drain holes are provided, they shall be at least 1-in. (25-mm) diameter (or slot width) on 8-ft (2,400-mm) centers or less. Stiffener rings on insulated tanks, where the stiffener rings function as insulation closures, shall have no drain holes or slots.

3.5.1.5 Weld joints. Welds joining stiffener components to the tank shell may cross vertical tank seam welds. Stiffener components may also cross vertical tank seam welds with the use of coping (i.e., rat hole) of the stiffener component at the vertical tank seam. Where the coping method is used, the required section modulus of the stiffening component and weld spacing must be maintained. Where required to perform the welds necessary to achieve the full design section properties of the stiffening component, welder access holes shall be provided (such as for butt joints in rolled WF beam sections where full penetration welds of the flange laying against the shell is required to achieve the minimum design section properties).

3.5.1.6 Stiffening rings as walkways. A stiffening ring or any portion of it that is specified as a walkway shall have a width not less than 28 in. (710 mm) clear of projections including the angle on the top of the tank shell. The clearance around local projections shall not be less than 24 in. (610 mm). Unless the tank is covered with a fixed roof, the stiffening ring (used as a walkway) shall be located 42 in. (1,100 mm) below the top of the curb angle and shall be provided with a standard railing on the unprotected side and at the ends of the section used as a walkway.

3.5.2 *Top shell girder*. Tanks without roofs shall have a top girder or angle having a minimum section modulus as determined by the formula:

$$S = 0.0001HD^2\left(\frac{P_{aw}}{18}\right) \tag{3-32)*$$

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 of $16t$ or $0.78(Rt)^{1/2}$ below and, if applicable, above the ring attachment to the shell), in cubic inches. When curb angles are attached to the top edge of the shell by butt welding, this

* For equivalent metric equation, see Sec. 3.14.

distance shall be measured from the underside of the horizontal leg of the angle.

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

D = the nominal diameter of the cylindrical portion of the tank shell, in feet

P_{aw} = weighted average wind pressure acting over the design height, in pounds per square foot, calculated using the design wind pressures, p_w, as calculated by Eq 3-5 for each of the height zones as defined in Table 2 that occur within the design height. For the top shell girder, the design height shall be taken as H.

R = the nominal radius of the cylindrical portion of the tank shell, in inches

t = the as-ordered shell thickness minus corrosion allowance at the girder or angle attachment location, in inches

3.5.2.1 Vertical leg of the angle. The total vertical leg of the angle may be used in the computations, provided that the vertical leg width does not exceed 16 times the angle thickness.

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

$$h = \frac{10.625 \times 10^6 t}{P_{aw}\left(\frac{D}{t}\right)^{1.5}} \qquad \text{(3-33)*}$$

Where:

h = the height of the cylindrical shell between the intermediate wind girder and the roof, top angle, or top wind girder, in feet

t = the average, as-ordered shell thickness minus corrosion allowance, in inches, for the vertical distance, h, unless otherwise specified

P_{aw} = weighted average wind pressure acting over the design height, in pounds per square foot, calculated using the design wind pressures, p_w, as calculated by Eq 3-5 for each of the height zones as defined in Table 2 that occur within the design height

* For equivalent metric equation, see Sec. 3.14.

For intermediate shell girders, the design height shall be taken as h.

3.5.3.1 Roof knuckle. Where an ornamental roof knuckle (torus transition) is specified, two-thirds of the transition height shall be added to the shell height except if either of the two following conditions exist.

3.5.3.1.1 If the roof knuckle is stiffened by radial stiffeners at a spacing of 7 ft (2.13 m) or less, and the radial stiffeners frame into a continuous circumferential stiffener equal in size to the first required intermediate stiffener, h may be measured from the bottom of the circumferential stiffener or the top of the straight cylindrical portion of the shell, whichever is greater.

3.5.3.1.2 If the roof knuckle is stiffened by radial rafters that extend through the torus region to the cylindrical shell at a spacing of 7 ft (2.13 m) or less at the top of the roof knuckle, h may be measured from the top of the straight cylindrical portion of the shell.

3.5.3.2 Procedure. In determining the maximum height of the 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 smaller than, the height of shell used in determining the average thickness. If h continues to calculate greater than the height of the shell used in determining the average thickness, no intermediate girder is required.

After establishing the location of the first intermediate girder, if required, repeat the previous 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 with 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 described previously or in the following alternative subsections.

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

$$W_{tr} = W\left(\frac{t_{uniform}}{t_{actual}}\right)^{2.5} \tag{3-34}$$

Where:

$t_{uniform}$ = the uniform, as-ordered thickness minus corrosion allowance into which the entire shell will be transformed, in inches

t_{actual} = the as-ordered thickness minus corrosion allowance of the shell course being transformed, in inches

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 midheight 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.

3.5.3.3 Intermediate girders. When intermediate girders are required, they shall be proportioned in accordance with the formula:

$$S = 0.0001hD^2\left(\frac{P_{aw}}{18}\right) \tag{3-35}^*$$

Where:

S = minimum required section modulus of the intermediate girder (including a portion of the tank shell for a distance of the lesser of $16t$ or $0.78(Rt)^{1/2}$ above and below the ring attachment to the shell), in cubic inches

The other symbols have been previously defined in this section.

3.5.4 *Tension and compression rings.* At junctures in shell plates where the meridional forces are discontinuous, a tension or compression ring is often required to resist the circumferential forces generated by the discontinuous membrane forces. Tension and compressive stresses are limited to those shown in Tables 5 and 6. To determine the stresses in the ring, the shell adjacent to the discontinuity may be assumed to participate for a maximum distance of $0.78(Rt)^{1/2}$ each way from the discontinuity point, where R is the normal radius of the tank section under consideration, in inches, and t is the thickness of the tank section under consideration, in inches.

* For equivalent metric equation, see Sec. 3.14.

Sec. 3.6 Roofs

3.6.1 *General requirements*. Roof supports and stiffeners for steel roofs, if used, shall be designed using the allowable stress design provisions of ANSI/AISC 360 with the following stipulations or exceptions:

3.6.1.1 Lateral support of rafters. Roof plates may be assumed to provide the necessary lateral support of roof rafters from the friction between the roof plates and the compression flange of the rafters, with the following exceptions: (a) trusses and open web joists used as rafters, (b) rafters having a nominal depth greater than 15 in. (381 mm), (c) rafters having a slope greater than 2 in 12, and (d) rafters supporting insulated cone roofs where both ends of the rafter are restrained against axial movement. When bracing is required to provide the necessary lateral support, bracing shall conform to the requirements of ANSI/AISC 360. Where the roof plate is incorporated into built-up rafter sections, the roof plate may be considered effective for a distance of 16t on each side.

3.6.1.2 Minimum roof slope. The minimum roof slope shall be ¾ in 12.

3.6.1.3 Column design. The maximum slenderness ratio, KL/r, for the roof-supporting columns shall be 175, where L is the laterally unsupported length and r is the radius of gyration of the column, both in inches. Columns shall be designed using the allowable stress design provisions of ANSI/AISC 360. Columns subject to lateral loads shall be designed as beam-columns.

3.6.1.4 Rafter design. Roof supports and stiffeners shall be designed using the allowable stress design provisions of ANSI/AISC 360 for ASTM A36/A36M when the roof design live load or roof design seismic load is 50 lb/ft^2 (2,400 N/m^2) or less. For roof design live loads or roof design seismic loads greater than 50 lb/ft^2 (2,400 N/m^2), design of roof supports and stiffeners may utilize higher allowable stresses when using material with minimum specified yield strength greater than ASTM A36/A36M. The aforementioned restriction on allowable stress does not apply to other roof support members such as columns and girders.

3.6.1.5 Placement of rafters. Roof rafters and trusses shall be placed above the MOL. No part shall project below the MOL.

3.6.1.6 Coating. Unless otherwise specified, priming or painting of contact surfaces between roof plates and rafters is not required.

3.6.1.7 Maximum spacing of roof supports and stiffeners. For supported roofs, maximum spacing of roof supports and stiffeners shall be:

$$L = \frac{2{,}575t}{\sqrt{W_{D+L}}} \leq 84 \tag{3-36}*$$

Where:

t = roof plate thickness, in inches

L = rafter centerline spacing at maximum radius, in inches

W_{D+L} = roof load (dead load plus greater of roof live load or snow load), in pounds per square foot

3.6.1.8. Seal-welding columns. Columns made from two or more structural shapes shall be seal welded their full height at all adjoining surfaces. Inaccessible surfaces under column bases shall be primed and painted before erection.

3.6.1.9 Supported-cone roofs. For supported-cone roofs requiring multiple columns and intermediate support girders, the rafters may be set directly on chord girders producing slightly varying rafter slopes. The slope of the flattest rafter shall conform to the specified roof slope.

3.6.1.10 Lateral support of columns. Details shall provide lateral support at the base and top of columns.

3.6.1.11 Deflection limit for roof plate. There is no deflection limit for roof plate that spans between structural supports.

3.6.2 *Self-supporting dome, umbrella, and cone roofs*. Self-supporting dome, umbrella, and cone roofs constructed of unstiffened carbon steel plates shall comply with the requirements of API 650 or 620. Self-supporting dome roofs constructed of aluminum shall comply with the requirements of ANSI/AWWA D108.

Sec. 3.7 Cylindrical Shell Plates

The thickness of cylindrical shell plates stressed by pressure of the tank contents shall be calculated by the formula:

$$t = \frac{2.6h_pDG}{sE} \tag{3-37}*$$

Where:

t = the required design shell plate thickness, in inches

* For equivalent metric equation, see Sec. 3.14.

h_p = the height of liquid from MWL to the bottom of the shell course being designed, in feet

D = the nominal tank diameter, in feet

G = product specific gravity (1.0 for water)

s = allowable design stress, in pounds per square inch

E = joint efficiency (see Table 11)

3.7.1 *Joints.* The longitudinal joints in adjacent circumferential courses may be either staggered or in alignment. Joints crossing each other shall be grooved and welded continuously through the intersections.

Sec. 3.8 Anchorage

3.8.1 *General.*

3.8.1.1 Required anchorage. For ground-supported flat-bottom reservoirs and standpipes, mechanical anchorage shall be provided when the wind or seismic loads exceed the limits for self-anchored tanks. Determination of limits for self-anchored tanks shall be in accordance with Sec. 3.8.6 and Sec. 3.8.8. Mechanical anchorage shall always be provided for elevated tanks.

Table 11 Weld design values—tank plate joints

	Efficiency-Percent	
Type of Joint	**Tension**	**Compression**
Double-groove butt joint with complete joint penetration	85	100
Double-groove butt joint with partial joint penetration and with the unwelded portion located substantially at the middle of the thinner plate	$85Z/T$ *	$100Z/T$ *
Single-groove butt joint with suitable backing strip or equivalent means to ensure complete joint penetration	85	100
Transverse lap joint with continuous fillet weld on each edge of joint	75	75
Transverse lap joint with continuous fillet weld on one edge of joint and an intermittent full thickness fillet weld on the other edge of joint	$75\frac{(1+X)}{2}$ †	$75\frac{(1+X)}{2}$ †
Transverse lap joint with fillet weld, or smaller, on either or both edges of the joint; welds either continuous or intermittent	$75\frac{(XW_1+YW_2)}{2t}$ ‡	$75\frac{(XW_1+YW_2)}{2t}$ ‡

* In which Z is the total depth of penetration from the surfaces of the plate (use the thinner plate if of different thicknesses) and T is the thickness of the plate (use the thinner plate if of different thicknesses).

† In which X is the ratio of the length of intermittent fillet weld to the total length of joint, expressed as a decimal.

‡ In which X and Y are the ratios of the lengths of intermittent welds W_1 and W_2, respectively, to the length of the joint, expressed as a decimal; W_1 and W_2 are the sizes of the welds on each edge of the joint, respectively (W_2 will be zero for a joint welded only on one edge); and T is the thickness of plate (use the thinner plate if of different thicknesses).

3.8.1.2 Spacing.

3.8.1.2.1 The maximum anchor spacing shall not exceed the following:

1. 6 ft (1.83 m) for single-pedestal tanks with a nominal base diameter less than 40 ft (12.2 m).
2. 10 ft (3.05 m) for single-pedestal tanks with a nominal base diameter equal to or greater than 40 ft (12.2 m) and for ground-supported flat-bottom tanks.

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

3.8.1.2.3 For ground-supported flat-bottom tanks and single-pedestal tanks, anchors 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 the uniform spacing. A special analysis is required at locations where tank openings or tank accessories interfere with more than two anchors.

3.8.1.3 Minimum number of anchor bolts. For single-pedestal tanks and for ground-supported flat-bottom tanks, the minimum number of anchor bolts shall be 6. For cross-braced multicolumn tanks, the minimum number of anchor bolts per column shall be 2.

3.8.2 *Types of anchors*. Anchors for elevated tanks shall be anchor bolts. Anchors for ground-supported flat-bottom tanks shall be anchor bolts or anchor straps.

3.8.3 *Unit stresses*.

3.8.3.1 Static loads. The allowable unit tension stress for ASTM A36/A36M, F1554, and A193/A193M, grade B7 anchor bolts shall be as given in Table 5. For anchor bolts made from materials other than those listed in Sec. 2.2.1 and for all other anchor types, the allowable stress for static load cases shall be the lesser of 0.4 times the published minimum yield stress or 0.25 times the published minimum ultimate stress of the material.

3.8.3.2 Wind loads. For wind load cases, the allowable unit tension stress for anchors shall be the basic allowable unit stress for the anchor with increases, if applicable, per Sec. 3.3.3.

3.8.3.3 Seismic loads. The allowable unit tension stress for seismic loads for ASTM A36/A36M, F1554-36, and F1554-55 (weldable) anchor bolts shall be as given in Sec. 3.3.3.2. For ASTM A193/A193M, grade B7; ASTM F1554-105;

and all other anchors, the allowable unit tension stress for seismic loads shall be the basic allowable unit stress with increases, if applicable, per Sec. 3.3.3.

3.8.3.4 Corroded condition. For all load cases, the stress levels in the anchor, anchor chair, anchor attachment, and embedment shall be evaluated in both the as-built condition and, if a corrosion allowance is specified, the corroded condition.

3.8.4 *Anchor requirements.*

3.8.4.1 Anchor bolts. Anchor bolts shall meet the following requirements:

1. Anchor bolts may be either upset or not upset. When upset anchor bolts are used, they shall be proportioned for the design loads using the corroded area at the root of the threads or at the not-upset bolt diameter, whichever is smaller.

2. When exposed to weather, a corrosion allowance of ¼ in. (6.35 mm) shall be applied to the root thread diameter of anchor bolts less than 1¼-in. (31.8-mm) diameter. Anchor bolts are considered exposed to weather when they are not protected by a sealed cover or enclosure or located inside the pedestal of single-pedestal tanks.

3. The minimum anchor-bolt diameter shall be 1 in. (25.4 mm).

4. Anchor-bolt embedment shall terminate in a head, nut, washer plate, or U-bolt.

5. The anchor bolt nuts shall be secured as follows:
    a. For ASTM A193/A193M, grade B7 bolts and ASTM F1554, grade 105 bolts, lock nuts shall be provided.
    b. For all other anchor bolts, lock nuts shall be provided or the threads shall be peened to prevent loosening of the nuts.

6. 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. (50.8-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).

7. 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.

8. Anchor bolts with a published minimum yield strength greater than 55,000 psi (379.2 MPa) shall be pretensioned using a hydraulic bolt tensioner to at least 80 percent of the design load.

9. The minimum published yield strength of anchor bolts made from materials other than those listed in Sec. 2.2.1 shall not exceed 70 percent of the minimum published tensile strength.

3.8.4.2 Anchor straps. Anchor straps shall meet the following requirements:

1. When anchor straps are not exposed to weather, the minimum anchor-strap thickness shall be ¼ in. (6.35 mm).

2. When anchor straps will be exposed to weather, a corrosion allowance of ¼ in. (6.35 mm) shall be added to the thickness of anchor straps and anchor plates with a required design thickness less than ½ in. (12.7 mm), and a corrosion allowance of ⅜ in. (9.53 mm) shall be added to the thickness of anchor straps and anchor plates with a required design thickness of ½ in. (12.7 mm) or greater. Anchor straps are considered exposed to weather when they are not protected by a sealed cover or enclosure.

3. Anchor-strap embedment shall terminate in an anchor plate welded to the bottom of the strap. The minimum thickness of the anchor plate shall be the as-provided thickness of the anchor strap. The minimum width and length of the anchor plate shall equal the width of the anchor strap. The thickness, width, length, and attachment of the anchor plate shall be sufficient to withstand, in the corroded condition, the anchorage design load within allowable stress limits of the anchor plate, or else, shear studs shall be added to the anchor strap in sufficient size and quantity to develop the remaining portion of the anchorage design load.

4. The design slope of the anchor strap from vertical shall not exceed 5.0°.

5. The design of the tank shell and attachment of the anchor strap to the tank shall consider the strap geometry, eccentricity, and weld configuration of the anchor strap attachment.

6. When anchor straps are used, the tank must be installed on a grouted foundation.

3.8.5 *Design of anchor chairs and attachments.*

3.8.5.1 Anchor-bolt chairs. Anchor-bolt chairs, if used, shall be configured and designed in accordance with AISI T-192, except as noted. The allowable local shell or pedestal stress shall be 20,000 psi (137.9 MPa), and the allowable stress on the effective throat of welds shall be in accordance with Sec. 3.12, both increased for wind or seismic loads, if applicable.

Other design procedures may be used, provided that they are based on a detailed analysis (e.g., finite element analysis) and account for local shell stresses. The local shell or pedestal stress for designs based on a detailed analysis shall be evaluated using the procedure given in Appendix 4 of ASME BPVC Sec. VIII, Div. 2, with S_m taken as the allowable tensile stress defined by this section.

Alternate anchor-bolt chair configurations may be used, provided that they are proven by test or calculation to meet the strength requirements of this section and that stresses of all components are limited to those specified in this section and, if applicable, increased in accordance with Sec. 3.3.3.

3.8.5.2 Other anchor attachments. Anchor attachments other than anchor-bolt chairs shall be designed so that stresses for all components are limited to those specified in this section and, if applicable, increased in accordance with Sec. 3.3.3.

3.8.5.3 Ductility. Anchor chairs and attachments shall be designed to withstand, without failure, the lesser of anchor yield capacity and the net uplift force determined by Eq 3-38. The anchor yield capacity shall be based on the corroded area at the root of the threads for anchor bolts or the corroded thickness for anchor straps.

$$P_{ab} = 4\left(\frac{4M_S}{ND_{ac}}\right) - \frac{(1 - 0.4A_v)(W_{DL} + W_{WL})}{N} \qquad \text{(3-38)*}$$

Where:

P_{ab} = net uplift force per anchor considering four times the seismic load, in pounds

M_S = seismic overturning moment, in foot-pounds

N = number of anchors

D_{ac} = diameter of anchor circle, in feet

W_{DL} = dead weight of structure (corroded condition) available to resist uplift, in pounds. For ground-supported flat-bottom tanks, W_{DL} shall be taken as the shell weight plus roof dead load reaction on shell.

W_{WL} = water weight available to resist uplift, in pounds. For ground-supported flat-bottom tanks, W_{WL} shall be taken as 0 lb.

A_v = vertical design acceleration from Sec. 13.4.3.3 or Sec. 13.5.4.3, stated as multiple (decimal) of g

* For equivalent metric equation, see Sec. 3.14.

3.8.6 *Design for resistance to base shear.*

3.8.6.1 Design loads. The net base shear to be resisted by the anchorage V_{NET} shall be that portion of the calculated base shear that exceeds the calculated frictional resistance. For elevated tanks, anchorage for base shear (sliding) need only be provided when V_{NET} exceeds zero. For ground-supported flat-bottom tanks, see Sec. 13.5.4.5. V_{NET} shall be resisted by only that portion of the shear anchorage system considered to be effective for any direction of ground motion.

3.8.6.2 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. Anchor straps shall not be considered as providing resistance to base shear.

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

1. Design of anchor embedment shall be in accordance with IBC, ACI 318, or ACI 349.
2. Provide at least 3-in. (76.2-mm) clear cover between the anchor and bottom of the foundation.
3. Installation of anchor bolts greater than 2 in. (50.8 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 both the as-built and, if corrosion allowance is specified, the corroded condition of the anchor and embedment.
5. Bond stress of the anchor shall not be considered in determining the load capacity of the anchor embedment.

3.8.8 *Design loads.*

3.8.8.1 Anchors. Anchors shall be designed for the maximum effect of the design uplift forces including the wind uplift force, P_W, and the seismic uplift force, P_S, considering applicable load combinations and allowable stress levels. The design uplift forces P_W and P_S represent the net uplift force to be resisted by the anchor after consideration for any reductions resulting from the dead weight of the structure and, for seismic loads on elevated tanks, the weight of the stored water. When the calculated net uplift force for both P_W and P_S results in a negative value, no uplift anchorage is required. For ground-supported flat-bottom tanks, Eq 3-40 only applies if the tank is mechanically anchored (see Sec. 13.5.4 to determine seismic anchorage requirements). For single-pedestal elevated tanks and mechanically anchored, ground-supported flat-bottom tanks, the design uplift forces P_W and P_S are calculated as follows:

$$P_W = \frac{4M_W}{ND_{ac}} - \frac{W_{DL}}{N} \tag{3-39}*$$

$$P_S = \frac{4M_S}{ND_{ac}} - \frac{(1 - 0.4A_v)(W_{DL} + W_{WL})}{N} \tag{3-40}*$$

Where:

P_W, P_S = design uplift force per anchor for wind and seismic loads, respectively, in pounds

M_W = wind overturning moment, in foot-pounds

The other symbols have been previously defined in this section.

3.8.8.2 Special load case for cross-braced multicolumn tanks. See Sec. 13.3.3.3 for a special load case for anchor bolts for cross-braced multicolumn tanks.

Sec. 3.9 Corrosion Allowance and Protection

3.9.1 *General.* If corrosion allowance is desired, it shall be specified for parts in contact with water and parts not in contact with water. The specified corrosion allowance shall be added to the required thickness determined by design, unless otherwise specified.

3.9.2 *Rolled shapes.* Corrosion allowance on structural sections shall be applied as a total per element (e.g., web or flange), unless otherwise specified.

3.9.3 *Bottom plates.* For bottom plates of ground-supported flat-bottom tanks, the specified corrosion allowance shall be added to the minimum thickness specified in Sec. 3.10.

3.9.4 *Anchorage.* Corrosion allowance requirements for anchors are given in Sec. 3.8.4.

3.9.5 *Tubular shapes.* Tubular shapes, when incorporated into the tank structure, shall be protected from corrosion by suitable coatings on the interior surfaces with access for maintenance, by hermetically sealing each member so that internal corrosion cannot occur, or by adding a corrosion allowance of not less than ⅛ in. (3.18 mm) to the design thickness.

* For equivalent metric equation, see Sec. 3.14.

Sec. 3.10 Minimum Thickness and Size

3.10.1 *Parts in contact with water.* All parts of the structure in contact with water when the tank is filled to the MWL shall have a minimum thickness of ¼ in. (6.35 mm), except as noted in Sec. 3.10.3 and except as follows. The minimum thickness of double butt-welded roof knuckles for ground-supported flat-bottom tanks with shell height less than 48 ft (14.6 m) and diameter not greater than 50 ft (15.2 m) may be 3⁄16 in. (4.76 mm).

3.10.2 *Parts not in contact with water.* The minimum thickness for all parts of the structure not in contact with water shall be 3⁄16 in. (4.76 mm), except as noted in Sec. 3.10.4, Sec. 3.10.6, and Sec. 3.10.7, and except as follows. For ground-supported flat-bottom tanks with cone roofs not in contact with water when the tank is filled to the MWL, the minimum thickness of roof plates shall be USS 7-gauge (4.55 mm) sheet. The minimum thickness for tubular columns and tubular struts shall be ¼ in. (6.35 mm).

3.10.3 *Cylindrical shell plates.* Cylindrical shell plates in contact with water when the tank is filled to the MWL shall have minimum thicknesses as shown in Table 12.

3.10.4 *Brace rods.* Bars used for wind bracing shall have a minimum diameter or width of ¾ in. (19 mm). Other shapes used for wind bracing shall have a total net cross-sectional area at least equal to that of a ¾ in. (19 mm) round bar.

3.10.5 *Overflow piping.* Steel overflow piping not in contact with water when the tank is filled to the MWL shall have a minimum thickness of 3⁄16 in. (4.76 mm).

Table 12 Minimum thickness of cylindrical shell plates in contact with water

Nominal Shell Diameter, *D*		Nominal Shell Height, *H*		Minimum Shell Thickness: Ground-Supported Flat-Bottom Tanks		Minimum Shell Thickness: Other Tanks	
ft	*(m)*	*ft*	*(m)*	*in.*	*(mm)*	*in.*	*(mm)*
$D \le 20$	$D \le 6.1$	All	All	3⁄16	4.76	¼	6.35
$20 < D \le 50$	$6.1 < D \le 15.2$	$H \le 48$	$H \le 14.6$	3⁄16	4.76	¼	6.35
$20 < D \le 50$	$6.1 < D \le 15.2$	$H > 48$	$H > 14.6$	¼	6.35	¼	6.35
$50 < D \le 120$	$15.2 < D \le 36.6$	All	All	¼	6.35	¼	6.35
$120 < D \le 200$	$36.6 < D \le 61.0$	All	All	5⁄16	7.94	5⁄16	7.94
$D > 200$	$D > 61.0$	All	All	⅜	9.52	⅜	9.52

3.10.6 *Rolled shapes*. For the purposes of Sec. 3.10, the controlling thickness of rolled shapes shall be taken as the mean thickness of the flange. The minimum web thickness of rolled shapes shall be 0.17 in. (4.32 mm).

3.10.7 *Anchors*. Minimum size and thickness requirements for anchors are given in Sec. 3.8.4.

3.10.8 *Butt-welded annulus*. When a butt-welded annulus is provided, the width of the butt-welded annulus measured from the inside of the shell shall not be less than 18 in. (457 mm).

Sec. 3.11 Joints

3.11.1 *Welded joints*. Joints between and connections to tank plates shall be welded.

3.11.2 *Bolted and pinned joints.* Bolts may be used for minor attachments and for column splices that carry mainly compression loads by bearing of the abutting edges. Bolted connections may be used for roof supports designed in accordance with Sec. 3.6, except that all connections to tank plates shall be welded. Finished bolts, or cold-rolled or finished pins, may be used for the attachment of tension rods, provided that the ends of the pins or bolts are fitted with nuts or welded washers. Bolts used in the attachment of tension rods shall have the threads burred outside the nuts to prevent easy removal of the nuts. Pins fitted with welded washers shall be welded at least 50 percent of the pin circumference with a 1/4-in. (6.35-mm) fillet weld.

Sec. 3.12 Weld Design Values

3.12.1 *Structural joints.* Welded structural joints shall be proportioned so that the stresses on the effective throat of the weld, exclusive of weld reinforcement, do not exceed the following percentages of the allowable tensile stress (Table 5) of the structural material joined.

3.12.1.1 Groove welds. Groove welds in tension, 85 percent; compression, 100 percent; shear, 75 percent.

3.12.1.1.1 Tension in welded bracing, 100 percent. See Sec. 8.2.1.2 for qualification of welding procedure specification and Sec. 11.4.5 for inspection.

3.12.1.2 Fillet welds. Fillet welds in transverse shear, 65 percent; longitudinal shear, 50 percent; varying shear around reinforcing pads, 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.

3.12.2 *Tank plate joints.* Weld design values for tank plate joints shall be as given in Table 11.

Sec. 3.13 Reinforcement Around Openings

Shell cutouts 4 in. (102 mm) and less in diameter having a welded-in neck need not be reinforced. Openings greater than 4 in. (102 mm) in diameter in the tank shell, suspended bottom, riser plating, and other locations that are subject to membrane tension stress caused by fluid pressure, where the thicknesses are established in accordance with the unit stresses given in Sec. 3.2, 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.

3.13.1 *Tank and riser plating.* The amount of reinforcement for an opening in the tank shell or riser plating, except for flush-type cleanout fittings, 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 plating perpendicular to the direction of the maximum stress and the required shell plate thickness, based on the permissible unit stress, the permissible joint efficiency, and corrosion allowance, if specified. 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). Effective reinforcement shall be placed symmetrically within a distance, perpendicular to the direction of maximum stress, in either direction from the centerline of the shell opening, equal to the maximum dimension of the hole in the shell plate in the direction 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 may be provided by the shell-plate joint efficiency and exclusive of any thickness specified for corrosion allowance, may be used as reinforcement area. See Sec. 3.13.2.5 for requirements pertaining to flush-type cleanout fittings.

3.13.2 *Fittings.* In the computation of the net reinforcing area of a fitting, such as a boilermaker's flange or a manhole saddle having a neck, 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 plate for a distance equal to four times the neck wall thickness or, if the neck wall thickness is not uniform within this distance, to the point of transition.
2. That portion lying within the shell plate thickness.

3. If the neck extends inwardly, that portion extending inward from the inside surface of the shell plate for a distance specified in item 1.

3.13.2.1 Required weld strength for fittings. The aggregate strength of the welding attaching a fitting to the shell plate 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.

3.13.2.2 Required weld strength for reinforcing plate. The aggregate strength of the welding attaching any intervening reinforcing plate to the shell plate shall at least equal the proportion of the forces passing through the entire reinforcement that is computed to pass through the reinforcing plate.

3.13.2.3 Effective weld. The attachment welding of 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 the inner peripheral welding shall be considered effective. The outer peripheral weld shall be of a size equal to the thickness of the shell plate or reinforcing plate, whichever is thinner, except that, when low-type nozzles or manholes are used with the reinforcing plate extending to a flat tank bottom, the size of that portion of the peripheral weld that attaches the reinforcing plate to the bottom plate shall conform to the requirements of Sec. 8.7. The inner peripheral welding shall be of sufficient size to carry the remainder of the loading.

3.13.2.4 Cut surfaces. Manhole necks, nozzle necks, reinforcing plates, and shell-plate openings that have sheared or oxyfuel gas-cut surfaces shall have such surfaces made uniform and smooth, with the corners rounded, except where the surfaces are fully covered by attachment welds.

3.13.2.5 Flush-type cleanout fittings. For ground-supported flat-bottom tanks that are provided with a flush-type cleanout fitting, the design, details, fabrication, inspection, and installation shall conform to the requirements of API 650 with the following exceptions:

1. Flush-type cleanout fittings are not permitted in materials from Sec. 14, category 3.

2. Cleanouts for tanks built in accordance with the design criteria of Sec. 3 are exempt from the preassembly and stress relief requirements of API 650, provided that no plate in the assembly is thicker than ⅝ in. (16 mm) and the opening is 12 in. (300 mm) or less in height. Cleanouts that exceed these limits shall conform

to the requirements of item 3 and shall be constructed of material from category 1 or 2 of Sec. 14.

3. Cleanouts for tanks conforming to Sec. 3 that exceed the limits of item 2, and cleanouts for tanks conforming to Sec. 14, shall be built in accordance with the requirements of API 650, including preassembly and stress relief.

Sec. 3.14 Equivalent Metric Equations

Metric equivalents of equations presented in Sec. 3 are as follows:

Equation Number	Equivalent Metric Equation	Variable	Units
3-1	$p_f = 0.7C_eC_tI_sp_g$	p_f, p_g	N/m^2
3-4	$q_z = \lambda_w\left[0.613K_zK_{zt}K_dV^2\right]$	q_z V	N/m^2 m/s
3-5	$p_w = q_zGC_f \geq 1{,}436C_f$	p_w, q_z	N/m^2
3-26	$A = 0.651 + 0.000555F_y$	F_y	MPa
3-27	$B = 0.355 - 0.000424F_y$	F_y	MPa
3-28	$D_1 = \dfrac{F_y}{937.69} + 0.196$	F_y	MPa
3-29	$D_2 = \dfrac{88.98}{F_y^{0.0573}} - 60.17$	F_y	MPa
3-31	$l_e = \dfrac{b}{t} = \dfrac{606}{\sqrt{f}}\left[1.0 - \dfrac{132}{\left(\frac{w}{t}\right)\sqrt{f}}\right]$	f w, b, t	MPa mm
3-32	$S = 0.06713HD^2P_{aw}$	S H, D P_{aw}	mm^3 m N/m^2
3-33	$h = \dfrac{8{,}025t}{P_{aw}\left(\frac{D}{t}\right)^{1.5}}$	h, D P_{aw} t	m N/m^2 mm
3-35	$S = 0.06713hD^2P_{aw}$	S h, D P_{aw}	mm^3 m N/m^2
3-36	$L = \dfrac{17.8t}{\sqrt{W_{D+L}}} \leq 2.13$	L t W_{D+L}	m mm N/m^2

Equation Number	Equivalent Metric Equation	Variable	Units
3-37	$t = \frac{4.9 h_p D G}{s E}$	t h_p, D s	mm m MPa
3-38	$P_{ab} = 4\left(\frac{4M_S}{ND_{ac}}\right) - \frac{9.81(1 - 0.4A_v)(W_{DL} + W_{WL})}{N}$	P_S M_S D_{ac} W_{DL} W_{WL}	N N-m m kg kg
3-39	$P_W = \frac{4M_W}{ND_{ac}} - \frac{9.81(W_{DL})}{N}$	P_W M_W D_{ac} W_{DL}	N N-m m kg
3-40	$P_S = \frac{4M_S}{ND_{ac}} - \frac{9.81(1 - 0.4A_v)(W_{DL} + W_{WL})}{N}$	P_S M_S D_{ac} W_{DL} W_{WL}	N N-m m kg kg

SECTION 4: SIZING AND DESIGN OF ELEVATED TANKS

Sec. 4.1 Standard Capacities

The standard capacities for elevated tanks shall be as given in Table 13.

Table 13 Standard capacities for elevated tanks

Volume		Volume	
US gal	***(m^3)***	***US gal***	***(m^3)***
50,000	(189.3)	400,000	(1,514.2)
60,000	(227.1)	500,000	(1,892.7)
75,000	(283.9)	750,000	(2,839.1)
100,000	(378.5)	1,000,000	(3,785.4)
150,000	(567.8)	1,500,000	(5,678.1)
200,000	(757.1)	2,000,000	(7,570.8)
250,000	(946.4)	2,500,000	(9,463.5)
300,000	(1,135.6)	3,000,000	(11,356.2)

Sec. 4.2 Heights for Elevated Tanks

The height of elevated-tank structures shall be measured from the top of the foundation to the BCL or to the MOL. The controlling height shall be specified.

Sec. 4.3 Standard Head Range

Where head range is immaterial, the head range shall be specified as "most economical." If a special head range is required, it shall be specified. Unless otherwise specified, a variation of ±2.5 ft (±0.76 m) in the head range is allowed to achieve maximum economy in design.

Sec. 4.4 Cross-Braced, Multicolumn Elevated Tanks

4.4.1 *Steel riser*. The steel riser shall be specified as wet or dry and shall be designed to withstand stress caused by the weight or the pressure of the tank and riser contents as well as the load imposed on the top of the riser by the tank and by any members supporting the tank. When considering axial load, the steel riser compressive stress shall not exceed the allowable stress calculated in accordance with Sec. 3.4.

4.4.1.1 Anchorage. Load-bearing steel risers shall be anchored to the foundation with a minimum of four anchor bolts. See Sec. 10.8 for grout requirements.

4.4.2 *Columns and struts*.

4.4.2.1 Column bases. Each column base shall have sufficient area to distribute the column load over the concrete foundations without exceeding the specified unit-bearing stress on the foundation. The connection of the column to the base plate shall provide for the maximum uplift, if the anchors are connected to the base plates and not to the column face.

4.4.2.2 Structural column splices. Column splices may be butt welded, or splice plates may be welded to both sections being joined. Column splices shall be designed to withstand the maximum possible uplift or at least 25 percent of the maximum compression if the columns are milled, whichever is greater. For unmilled columns, column splices shall be designed to withstand the maximum possible uplift or 100 percent of the maximum compression, whichever is greater.

4.4.2.3 Bottom struts. Bottom struts of steel or reinforced concrete shall be provided where necessary to distribute the horizontal reactions at the bases of the columns. These shall consist of struts connecting the foundation piers or of structural members connecting the lower ends of the columns.

4.4.3 *Tension members carrying wind and seismic loads.* Tension members shall be designed to resist the wind load and the seismic load if the latter is applicable. Bracing connections shall be designed to develop the minimum published yield capacity of the tension bracing member. It is not necessary to combine wind and seismic loads, but it is necessary to design for the maximum force produced by either load case. Threaded tension members designed to resist seismic load shall have upset or enlarged ends.

If the projected lines of action of tension members do not meet the projected line of action of strut members at the line of action of the columns, and the resulting eccentricity exceeds 0.15 times the diameter of the column, proper allowance shall be made. When eccentricity is equal to or greater than 0.15 times the diameter, in addition to satisfying the requirements of Sec. 3.4, the combined axial plus bending stress in the column caused by either wind or seismic loads shall be limited to 0.80 times the yield strength for tension stress and $1.5F_L$ for compressive stress, without applying the one-third allowable stress increase of Sec. 3.3.3.

4.4.3.1 Prestressing. Diagonal tension members shall be prestressed before the tank is filled to reduce sagging after the tank is filled. Such prestressing shall not be given consideration in the design of the members. Unless otherwise specified, tightening shall be accomplished with turnbuckles in diagonal tension members. Heat shrinking or other prestressing devices may be used when available turnbuckle sizes are exceeded.

4.4.3.2 Pin-connected tension member. In pin-connected tension members other than forged eyebars, the net section across the pinhole, transverse to the axis of the member, shall not be less than 125 percent of the net section of the body of the member, and the net section beyond the pinhole, parallel with the axis of the member, shall not be less than 62.5 percent of the net section of the body of the member.

4.4.4 *Horizontal girders.* For elevated tanks with inclined or battered columns connecting to the tank shell, a horizontal girder shall be provided to resist the horizontal component of the column loads. This girder shall be proportioned to withstand safely as a ring girder the horizontal inward component of the column load and other horizontal shear loads on the top column section.

4.4.4.1 Lines of action. If the lines of action of the horizontal girder, top column section, and tank shell do not meet at the work point, then provisions shall be made in the design of each of these for stresses resulting from any eccentricity.

4.4.4.2 Balcony and railing. If the horizontal girder is used as a balcony, it shall be a minimum of 24 in. (610 mm) in width and shall be provided with a railing of at least 42 in. (1,067 mm) in height.

Sec. 4.5 Tank Plates

4.5.1 *Shapes*. Plates for elevated tank bottoms, shells, and roofs may be any desired shape.

4.5.2 *Maximum unit stress*. Plates subject to complete stress analysis shall be designed in accordance with the requirements of Sec. 3. The maximum unit stress shall be reduced for the joint efficiencies set forth in Table 11. Such plates include those not stressed by the concentrated reactions of supporting members or riser pipes.

4.5.3 *Cross-braced multicolumn tanks*. Plates not susceptible to complete stress analysis shall meet the following requirements:

4.5.3.1 Shell and bottom. The shell and bottom shall be designed on the basis of 15,000 psi (103.4 MPa) maximum unit stress, reduced for the joint efficiency set forth in Table 11 and making allowances for the following:

1. The hoop stresses caused by the weight or pressure of the tank contents.
2. The stresses in the cylindrical shell and ellipsoidal bottom, considering them acting together as a circular girder supported by the column reactions and subjected to torsion because of the portions projecting outward and inward from the chords connecting the columns.
3. The horizontal inward component of the pull from the tank bottom (in conical or segmental bottoms) causing compression in the tank shell.
4. Stresses from any other causes.

The cylindrical shell and bottom shall be designed assuming that the cylindrical tank shell is uniformly supported on its entire lower circumference. For this design case, the thicknesses of the ring of the cylindrical shell to which the columns attach and the bottom shall be increased, if necessary, so that the maximum calculated unit stress shall not exceed 11,000 psi (75.8 MPa), reduced by the joint efficiency.

4.5.3.2 Welded-column connections. Welded-column connections to the tank shall be designed on the basis of values given in Sec. 3.12.1 using 15,000 psi (103.4 MPa) for plate tension and the value shown in Table 8 for plate shear.

4.5.4 *Tank bottom*. In designing bottoms of double curvature, consideration shall be given to the possibility of governing compressive stresses.

Sec. 4.6 Pedestal Tanks

4.6.1 *Pedestal supports.* Pedestal supports may be cylindrical, conical, doubly curved, folded or pressed plate, or any combination thereof. Adequate stiffening shall be provided to exclude ovalling or gross buckling of the pedestal. Openings other than manholes through the support pedestal shall be minimized and properly distributed to provide adequate shear transfer and vertical load transfer to the foundation. Unless a detailed analysis is performed, the following limitations shall apply:

1. Total perimeter removal when measured at the top of the opening shall not exceed 10 percent of the pedestal circumference.
2. Other than manholes of 36 in. (910 mm) or less in width, sizes of openings when measured at the top of the opening shall be limited to 20° of the pedestal circumference or a versine of 1.0 ft (300 mm), whichever is less.
3. Centerline spacing of adjacent openings shall be at least twice the sum of the width of the openings.

4.6.2 *Eccentric load.* Slender pedestals shall be designed with consideration of the P-delta effect. Slender pedestal tanks are defined as those having a lateral deflection greater than 0.02 times the minimum pedestal radius.

Sec. 4.7 Tank Stability Against Overturning

The tank stability against overturning shall be checked as outlined in Sec. 12.4 and Sec. 12.5 with the tank empty and maximum wind load. If applicable, stability under seismic loading shall also be checked with the tank full.

Sec. 4.8 Lateral Load Distribution

Combined pedestal and column-supported towers shall distribute shear and moment to the foundation in relation to the relative stiffness of each component.

SECTION 5: ACCESSORIES FOR ELEVATED TANKS

Sec. 5.1 Steel Riser

In localities where freezing temperatures do not occur, a small diameter steel riser may be specified. In other locations and unless a small pipe is specified, a steel riser not less than 36 in. (910 mm) in outside diameter (OD) shall be furnished.

5.1.1 *Safety grill.* A safety grill at the top of the riser shall be provided when specified. A safety grill is intended to prevent a person from falling down the riser and shall be exempt from the design loads specified in Sec. 3.1.8. When a safety grill is used in the top of the riser during erection, it shall be removed if the tank is located in climates where freezing is likely to occur. When grills are left in place, they shall be provided with a hinged door that is at least 18 in. × 18 in. (457 mm × 457 mm) in size.

5.1.2 *Expansion joint*. Where the riser is non-load-bearing, flexibility to accommodate differential movements of the tank and riser foundation must be included. This flexibility may be provided by an expansion joint or by riser layouts that have sufficient offset to be axially deformed without overstressing the riser, tank, or foundation.

Sec. 5.2 Pipe Connection

The size of the connecting pipe and the point where the pipe connects to the riser bottom shall be specified.

5.2.1 *Silt stop*. If a removable silt stop is specified, it shall be at least 6 in. (152 mm) high, and the fitting or piping connection shall be flush with the riser floor when the stop is removed. If a removable silt stop is not required, the connecting pipe shall extend at least 6 in. (152 mm) (and preferably about 2.50 ft [0.79 m]) above the riser floor.

5.2.2 *Inlet protection*. On risers 36 in. (910 mm) in diameter or larger, the inlet pipe shall be protected against the entry of foreign materials dropping from above. This shall be done by terminating the inlet pipe or the top of the silt stop pipe with a tee, with the "run" of the tee placed horizontally, or by placing over the silt stop or inlet pipe a circular plate 8 in. (203 mm) larger in diameter than the pipe and located horizontally above the end of the pipe or silt stop a distance equal to the diameter of the pipe. The circular plate assembly shall attach to the pipe or riser and be removable. Adequate clearance shall be provided between the ends of the tee or from the edge of the circular plate to the wall of the riser pipe to permit proper flow of water through the inlet pipe. Pipe connections to the riser shell are permitted, as long as adequate protection against freezing has been provided.

Sec. 5.3 Overflow

The tank shall be equipped with an overflow to protect the tank from overpressure and overload. The type and size of the overflow shall be specified. If a stub overflow is specified, it shall project at least 12 in. (304 mm) beyond the tank shell. For tanks with horizontal balcony girders, the overflow shall be extended to discharge below the balcony. If an overflow to ground is specified, it shall terminate near grade, and the discharge shall be directed away from the foundation and over a drainage inlet structure or splash block. The overflow to ground shall be located such that it will not be obstructed by snow or ground clutter.

Unless otherwise specified, the overflow may be external or internal. An internal overflow is defined as an overflow with piping inside the tank container. The consequences of an overflow failure, which can empty the tank contents, shall be considered when an internal overflow is provided.

The overflow shall originate at the top in a weir box or other appropriate type of intake. The top angle of the shell shall not be cut or partially removed. The overflow shall have a capacity at least equal to the specified inlet rate, with a head above the lip of the overflow of not more than 12 in. (304 mm) for side-opening overflows and not more than 6 in. (152 mm) for other types of overflows. Unless otherwise specified, the outlet of the overflow pipe shall be covered with a coarse, corrosion-resistant screen equivalent to ⅜ in. (9.5 mm) or larger mesh, or a flap valve.

Unless otherwise specified, the overflow pipe shall be steel pipe, with screwed or welded connections if less than 4 in. (102 mm) in diameter, or with flanged or welded connections if 4 in. (102 mm) in diameter or larger. The overflow pipe shall be supported at proper intervals with suitable brackets.

Sec. 5.4 Access

5.4.1 *General.* Ladders, stairs, platforms, rails, access openings, and safety devices shall comply with OSHA standards. Requirements for personal fall arrest systems, rest platforms, roof–ladder guardrails, and other safety devices that are in excess of the requirements given by OSHA shall be specified.

5.4.2 *Ladders.*

5.4.2.1 General. Ladders shall have side rails not less than 2 in. × ⅜ in. (51 mm × 9.5 mm), with a spacing between the side rails of not less than 16 in. (406 mm) and rungs not less than ¾ in. (19 mm) round or square, consistently spaced 10 in. (254 mm) to 14 in. (355 mm) apart on centers. Ladders shall not, in any place, have a backward slope. Ladders with a single point of connection, including rolling ladders, shall not be used. Skid-resistant rungs shall be provided when specified.

5.4.2.2 Tower ladder. For multicolumn tanks, a tower ladder shall be provided for access from a point 10 ft (3.0 m), or as specified, above grade to the horizontal balcony girder or the tank ladder, if no balcony girder is provided. For single-pedestal tanks, an inside tower ladder shall be provided for access from grade to a platform directly beneath the tank.

5.4.2.3 Access tube ladder. For single-pedestal tanks, an access tube and ladder shall be provided for access from the platform directly beneath the tank to the roof.

5.4.2.4 Outside tank ladder. For multicolumn tanks, an outside tank ladder shall be provided for access from the horizontal balcony girder or the tower ladder, if no balcony girder is provided, to the roof.

5.4.2.5 Access to roof hatches and vents. Access to roof hatches and vents shall be provided. Such access shall be from the outside tank ladder for multicolumn tanks and from the access tube ladder for single-pedestal tanks. Access shall comply with the following:

1. Where roof slope is 4 in 12 or greater, a ladder or stairs shall be provided.
2. Where roof slope is less than 4 in 12, a guardrail system, a personal fall arrest system, a travel restraint system, or work rule shall be provided in accordance with OSHA requirements.

5.4.2.6 Inside tank ladder. When specified, an inside tank ladder shall be provided for access from the roof to the bottom of the tank.

5.4.3 *Roof openings.*

5.4.3.1 Above MWL. An opening shall be provided above the MWL. It shall have a minimum dimension of 24 in. (610 mm), or as required by OSHA, if used to access an inside tank ladder, and shall be provided with a suitable hinged cover and a hasp to permit locking. The opening shall have a curb of at least 4 in. (102 mm) high, and the cover shall have a downward overlap of at least 2 in. (51 mm).

5.4.3.2 Tank center. An opening with a removable cover having an opening dimension or diameter of at least 20 in. (500 mm) and a 4-in. (102-mm) minimum height neck shall be provided at, or near, the center of the tank. The opening may be used as a tank vent opening, provided that the vent is removable. Where conveniently accessible from an outside balcony or platform, a shell manhole may be substituted for the additional opening. If properly designed, the shell manhole may be placed below the MWL.

5.4.4 *Steel riser manhole.* Risers 36 in. (910 mm) in diameter and greater shall contain a manhole about 3 ft (0.91 m) above the base of the riser. The manhole shall not be less than 12 in. × 18 in. (305 mm × 457 mm) in size, and the opening shall be reinforced in accordance with Sec. 3.13.

Sec. 5.5 Vent

Tanks equipped with roofs shall have a vent above the MWL, which shall have a capacity to pass air so that, at the maximum flow rate of water, either entering or leaving the tank, excessive pressure will not develop. The overflow pipe shall not be considered a tank vent.

5.5.1 *Location.* One tank vent shall always be located near the center of the roof, even if more than one tank vent is required. For tanks with centrally located access tubes, a reasonable offset of the vent is permissible.

5.5.2 *Screening.* The vent shall be designed and constructed to prevent the entrance of birds or animals. When the vent is provided with screening against insects, 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. The screens or relief mechanism shall not be damaged by the occurrence and shall return automatically to operating position after the blockage is cleared.

Sec. 5.6 Antennas and Related Equipment

When specified, loads from antennas and related equipment shall be included in the design of the tank, support structure, and foundation. Related topics are covered in Appendix A, Commentary for Welded Carbon Steel Tanks for Water Storage.

Sec. 5.7 Galvanic Corrosion

Dissimilar metals (e.g., stainless steel, copper, and brass) 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.

SECTION 6: SIZING OF GROUND-SUPPORTED STANDPIPES AND RESERVOIRS

The committee vacated this section during a previous revision cycle.

SECTION 7: ACCESSORIES FOR GROUND-SUPPORTED STANDPIPES AND RESERVOIRS

Sec. 7.1 Flush-Type Cleanouts

If flush-type cleanouts are specified, they shall comply with Sec. 3.13.2.5.

Sec. 7.2 Pipe Connections

The size of the connecting pipe, the point where the pipe connects to the tank, and the piping loads shall be specified. The tank shall be designed for the reactions imposed by specified piping loads. The piping and supports shall be designed so as not to impart significant mechanical loading on the attachment to the tank. (See also Sec. 13.7.)

7.2.1 *Silt stop*. If a removable silt stop is specified, it shall be at least 4 in. (102 mm) high, 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 specified, the fitting or connecting pipe, or both, shall extend above the floor at least 4 in. (102 mm).

7.2.2 *Shell connections*. Shell connections may be specified, provided that adequate provisions are made to protect the pipe from freezing and provided that the piping is sufficiently flexible. Piping must be flexible enough to accommodate shell rotation and deflections due to elastic growth caused by hydrostatic pressure, seismic movements, and settlement in the tank or piping system. See Sec. 13.7 for potential seismic movements.

7.2.3 *Bottom connections.* Bottom connections shall comply with Sec. 13.7.2 as a minimum.

Sec. 7.3 Overflow

The tank shall be equipped with an overflow to protect the tank from overpressure and overload. The type and size of the overflow shall be specified. If a stub overflow is specified, it shall project at least 12 in. (304 mm) beyond the tank shell. If an overflow to ground is specified, it shall terminate near grade, and the discharge shall be directed away from the foundation and over a drainage inlet structure or splash block. The overflow to ground shall be located such that it will not be obstructed by snow or ground clutter.

Unless otherwise specified, the overflow may be external or internal. An internal overflow is defined as an overflow with piping inside the tank container.

The consequences of an overflow failure, which can empty the tank contents, shall be considered when an internal overflow is provided.

The overflow shall originate at the top in a weir box or other appropriate type of intake. The top angle of the tank shell shall not be cut or partially removed. The overflow shall have a capacity at least equal to the specified inlet rate, with a head above the lip of the overflow of not more than 12 in. (304 mm) for side-opening overflows and not more than 6 in. (152 mm) for other types of overflows. Unless otherwise specified, the outlet of the overflow pipe shall be covered with a coarse, corrosion-resistant screen equivalent to ⅜ in. (9.5 mm) or larger mesh, or a flap valve.

Unless otherwise specified, the overflow pipe shall be steel pipe, with screwed or welded connections if less than 4 in. (102 mm) in diameter, or with flanged or welded connections if 4 in. (102 mm) in diameter or larger. The overflow pipe shall be supported at proper intervals with suitable brackets.

Sec. 7.4 Access

7.4.1 *General.* Ladders, stairs, platforms, rails, access openings, and safety devices shall comply with OSHA standards. Requirements for personal fall arrest systems, rest platforms, roof–ladder guardrails, and other safety devices that are in excess of the requirements given in OSHA shall be specified.

7.4.2 *Ladders.*

7.4.2.1 General. Ladders shall have side rails not less than 2 in. × ⅜ in. (51 mm × 9.5 mm), with a spacing between the side rails of not less than 16 in. (406 mm) and rungs not less than ¾ in. (19 mm) round or square, consistently spaced 10 in. (254 mm) to 14 in. (355 mm) apart on centers. Ladders shall not, in any place, have a backward slope. Ladders with a single point of connection, including rolling ladders, shall not be used. Skid-resistant rungs shall be provided when specified.

7.4.2.2 Outside tank ladder. An outside tank ladder shall be provided for access from a point 10 ft (3.0 m), or as specified, above the tank bottom to the roof or roof ladder. The location of the ladder shall be specified.

7.4.2.3 Access to roof hatches and vents. Access to roof hatches and vents shall be provided. Such access shall be from the outside tank ladder and shall comply with the following:

1. Where roof slope is 4 in 12 or greater, a ladder or stairs shall be provided.

2. Where roof slope is less than 4 in 12, a guardrail system, a personal fall arrest system, travel restraint system, or work rule shall be provided in accordance with OSHA regulations.

7.4.2.4 Inside tank ladder. When specified, an inside tank ladder shall be provided for access from the roof to the bottom of the tank.

7.4.3 *Roof openings.*

7.4.3.1 Ladder. A roof opening with hinged cover and hasp for locking shall be provided near the outside tank ladder, or roof ladder if provided. The opening shall have a minimum dimension of 24 in. (610 mm), or as required by OSHA, if used to access an inside tank ladder. The opening shall have a curb at least 4 in. (102 mm) high, and the cover shall have a downward overlap of at least 2 in. (51 mm).

7.4.3.2 Roof center. An additional opening with a removable cover having an opening dimension or diameter of at least 20 in. (510 mm) and a 4-in. (102-mm) minimum height neck shall be provided at, or near, the center of the tank. The opening may be used as a tank vent opening, provided that the vent is removable.

7.4.4 *Shell manholes.* Two shell manholes shall be provided in the first ring of the tank shell. The location of the manholes shall be specified. If any access cover weighs more than 50 lb (22.7 kg), a hinge or davit shall be provided. At least one manhole shall be circular with a minimum diameter of 30 in. (760 mm). Other manholes may be circular, 24 in. (600 mm) in diameter, or elliptical, 18 in. × 22-in. (450 mm × 550-mm) minimum size. The shell plate where the manholes are located shall be reinforced to comply with Sec. 3.13, and portions of the manholes, including reinforcing of the neck, the bolting, and the cover, shall be designed to withstand the weight and pressure of the tank contents.

Sec. 7.5 Vent

Tanks equipped with roofs shall have a vent above the MWL, which shall have a capacity to pass air so that, at the maximum flow rate of water, either entering or leaving the tank, excessive pressure will not develop. The overflow pipe shall not be considered a tank vent.

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

7.5.2 *Screening.* The vent shall be designed and constructed to prevent the entrance of birds or animals. When the vent is provided with screening against insects, 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. The screens or relief mechanism shall not be damaged by the occurrence and shall return automatically to operating position after the clogging is cleared.

Sec. 7.6 Antennas and Related Equipment

When specified, loads from antennas and related equipment shall be included in the design of the tank and foundation. Related topics are covered in Appendix A, Sec. A.5.6.

Sec. 7.7 Galvanic Corrosion

Dissimilar metals (e.g., stainless steel, copper, and brass) more noble than carbon steel and installed inside the tank below the MWL shall be electrically isolated from carbon steel tank components to which they are attached.

SECTION 8: WELDING

Sec. 8.1 Definitions and Symbols

Welding terms used in this standard shall be interpreted according to the definitions given in AWS A3.0M/A3.0. Symbols used on construction drawings shall conform to those shown in AWS A2.4 unless detailed weld sections are shown.

Sec. 8.2 Qualification of Welding Procedures, Welders, and Welding Operators

Tanks built according to this standard may be welded by any welding process that complies with the qualification requirements of ASME BPVC Sec. IX, AWS B2.1/B2.1M, or AWS D1.1/D1.1M. The welding may be performed manually, semiautomatically, automatically, or by machine welding. Single-pass electrogas deposits greater than ¾ in. (19 mm) and electroslag welds in all thicknesses may be used, provided that they meet requirements of Clause 4, Part D of AWS D1.1/D1.1M. Impact testing, when required, shall be conducted at the lowest one-day mean temperature (see Figure 6) plus 15°F (8.3°C). In no case shall the impact temperature be greater than 50°F (10°C).

8.2.1 *Qualification of welding procedure specifications*. Each welding procedure specification (WPS) shall be qualified by the employer of the welding personnel in accordance with the rules in ASME BPVC Sec. IX, AWS D1.1/D1.1M, or AWS B2.1/B2.1M. The use of an AWS B2.1/B2.1M standard welding procedure specification (SWPS) is not allowed without qualification by the employer in accordance with ASME BPVC Sec. IX, Article V, QW-510. Tests of the procedures shall be performed to determine that the weldment proposed for construction is capable of providing the required properties for its intended application. Production weld test plates need not be made.

8.2.1.1 Partial-joint penetration weld. Qualification of welding procedure specifications for partial-joint penetration welds (see Sec. 8.4.2) shall include an additional reduced-section tension test to demonstrate that the welding procedure and joint configuration to be used will produce a welded joint with strength not less than two-thirds the published minimum tensile strength of the base metal.

8.2.1.2 Tension bracing designed utilizing 100 percent efficiency. Splice welds shall be made using low-hydrogen welding processes. Qualification of welding procedure specifications shall include a full-size proof test to $\frac{4}{3}$ times the published minimum yield strength of the bracing member. The same welding processes and the same type bracing material that is used for the full-size proof test shall be used in production. Welding procedures and brace splice joint details qualified on one size brace will qualify smaller brace sizes.

8.2.1.3 Materials. All materials listed in Sec. 2.2.3, Sec. 2.2.4, Sec. 2.2.5, Sec. 2.2.11, and Sec. 14, except ASTM A517/A517M, shall be accepted in P-Number 1, group 1, 2, or 3 material grouping of ASME BPVC Sec. IX. ASTM A517/A517M shall be accepted in P11B group 3 grouping.

8.2.1.4 Acceptable welding procedure specifications. Acceptable welding procedure specifications shall be identified for each joint to be welded.

8.2.1.5 Providing welding procedure specifications. When specified, welding procedure specifications and supporting procedure qualification records shall be provided.

8.2.2 *Qualification of welders and welding operators*. All welders assigned to manual or semiautomatic welding and welding operators assigned to automatic or machine welding shall be tested to demonstrate their ability to make acceptable welds. The tests shall be as required in ASME BPVC Sec. IX, AWS D1.1/D1.1M, or AWS B2.1/B2.1M.

8.2.2.1 Test records. The records of such tests shall be as follows: Each welder or welding operator shall be assigned an identifying number, letter, or symbol. A certified record of the welders and welding operators employed, showing the date and results of tests and the identifying mark assigned to each, shall be maintained.

Sec. 8.3 Weld Joint Records

The identifying mark assigned to each welder or welding operator (Sec. 8.2.2.1) shall be stamped either by hand or by machine using a low-stress die on all tanks adjacent to and at intervals of not more than 3 ft (0.9 m) along the welds made by a welder or welding operator. Alternatively, a record of welders employed on each joint may be maintained.

Sec. 8.4 Butt Joints

8.4.1 *Butt joints subject to primary stress due to weight or pressure of tank contents.* Butt joints subject to primary stress, such as longitudinal joints of cylindrical tank shells and roof knuckles, and all joints below the point of support in suspended bottoms of elevated tanks, shall have complete joint penetration welds. Such welds may be double welded or single welded with a backing strip or equivalent means to ensure complete joint penetration.

8.4.2 *Butt joints subject to secondary stress.* Butt joints subject to secondary stress, such as circumferential joints of cylindrical tank shells, shall be welded as follows:

1. For base metals ⅜ in. (9.5 mm) or less in thickness, joints shall be welded using complete joint penetration welds.

2. For base metals of thickness greater than ⅜ in. (9.5 mm), joints shall be double welded and shall be welded using either partial joint penetration or complete joint penetration welds, unless complete joint penetration welds are specified. In addition, complete joint penetration welds shall be provided for a distance of at least 3 in. (76 mm) on each side of intersecting joints. In partial joint penetration welds, the effective throat shall exceed two-thirds of the thickness of the thinner plate. The unwelded portion shall be located near the center of the thinner plate. Alternatively, complete joint penetration welds used in lieu of partial joint penetration welds shall be made using filler metal compatible with the base metal and having a published minimum tensile strength not less than two-thirds the published minimum tensile strength of the base metal.

Sec. 8.5 Lap Joints

8.5.1 *Lap joints subject to primary stress due to weight or pressure of tank contents.* Lap joints subject to primary stress, such as longitudinal joints of cylindrical tank shells and all joints below the point of support in suspended bottoms of elevated tanks, shall have continuous fillet welds on both edges of the joint.

8.5.2 *Lap joints subject to secondary stress.* Lap joints subject to secondary stress, such as circumferential joints of cylindrical tank shells and roof knuckles, shall be welded on both sides with continuous fillet welds. They shall be designed to develop an efficiency of at least 50 percent based on the thickness of the thinner plate joined.

Sec. 8.6 Flat Tank Bottoms Resting Directly on Grade or Foundation

Flat tank bottoms shall be built by one of the following two alternative methods of construction.

8.6.1 *Lap joint construction*. Bottom plates shall be welded on the top side only, with continuous fillet welds on all seams.

Plates under the bottom ring of cylindrical shells shall have the outer ends of the lap joints depressed to form a smooth bearing for the shell plates.

Three-plate laps in tank bottoms shall be at least 12 in. (305 mm) from each other and from the tank shell. The maximum thickness for lap-welded bottoms shall be ⅜ in. (9.5 mm).

8.6.2 *Butt joint construction*. Butt joints may be welded from the top side, using a suitable backing strip or equivalent means to ensure at least 90 percent joint fusion. The three-plate joints in the tank bottoms shall be at least 12 in. (305 mm) from each other and from the tank shell.

Sec. 8.7 Shell-to-Bottom Joint

For cylindrical shells with flat bottoms, the bottom edge of the lowest course shell plates and the bottom plates shall be joined by continuous fillet welds on both sides of the shell plate. The maximum size of each fillet weld shall be ½ in. (13 mm). The minimum size of each fillet weld shall be either the nominal thickness of the thinnest plate joined or the size given in Table 14, whichever is larger. The required fillet sizes have no reference to any requirements for minimum plate thicknesses. The bottom plate shall extend outside the tank shell a distance of at least 1 in. (25 mm) beyond the toe of the weld. Where seismic uplift may occur, the tank design shall be checked to determine whether minimum weld size is adequate.

Table 14 Minimum size of fillet weld—shell-to-bottom joint

Thickness of Shell Plate				Minimum Size of Fillet Weld	
Minimum		Maximum			
in.	*(mm)*	*in.*	*(mm)*	*in.*	*(mm)*
3⁄16	(4.7)	<¼	(<6.4)	3⁄16	(4.7)
¼	(6.4)	¾	(19.0)	¼	(6.4)
>¾	(>19.0)	1¼	(31.8)	5⁄16	(7.9)
>1¼	(>31.8)	2	(50.8)	⅜	(9.5)

Sec. 8.8 Shell Plates

Longitudinal joints in adjacent circumferential courses may be either staggered or in alignment. Where joints cross each other, the first weld shall be grooved so that the second weld may be welded continuously through the intersection.

Sec. 8.9 Tubular Column and Strut Sections

Circumferential joints in tubular columns and strut sections shall have complete joint penetration welds. Columns more than 30 in. (750 mm) in diameter may use two-sided or one-sided welds using a backup bar or a one-sided pipe joint detail designed to achieve 100 percent penetration and fusion.* The longitudinal joints shall be butt joints, welded at least from the outside, but need not have complete joint penetration, provided that the total depth of weld, including not more than 1⁄16 in. (1.6 mm) of the reinforcement, is at least equal to the thickness of the plate.

Sec. 8.10 Steel Risers

Longitudinal joints in risers shall have complete joint penetration welds. Circumferential joints in risers that are also used as a column to support substantial vertical loads shall be butt joints, welded for complete joint penetration. For risers on suspended-bottom tanks where the riser supports only nominal vertical loads, the circumferential joints may be partial-penetration butt joints with an effective throat not less than two-thirds of the thickness of the thinner plate, or double-welded lap joints with full fillet welds on both edges.

Sec. 8.11 Roof Plates

8.11.1 *Roofs not subject to hydrostatic pressure.* In roofs not subject to hydrostatic pressure under normal operation from tank contents, lap joints may be welded on the top side only, with continuous fillet welds. Butt joints shall be single-groove welds, using suitable backing or equivalent means to ensure at least 90 percent joint penetration.

8.11.2 *Roofs subject to hydrostatic pressure.* In roofs subject to hydrostatic pressure under normal operation from tank contents, roof-plate joints shall be designed to conform to the joint efficiency values given in Sec. 3.12.2. The roofs may use lap joints welded with continuous double-fillet welds or butt joints with complete joint penetration welds to suit the strength requirements.

* Exception: When in contact with tank contents, accessible columns shall be seal welded from both sides.

Sec. 8.12 Maximum Thickness of Material to Be Welded

8.12.1 *Lap joints.*

8.12.1.1 Primary stress. The maximum thickness of material to be used for lap joints subject to primary stress because of the weight or pressure of tank contents, such as longitudinal joints of cylindrical tank shells and joints below the points of support in elevated tanks, shall be ½ in. (13 mm).

8.12.1.2 Secondary stress. The maximum thickness of material to be used for lap joints subject to secondary stress, such as circumferential joints of cylindrical tank shells and roof knuckles, shall be ⅝ in. (16 mm).

8.12.1.3 Flat tank bottoms. The maximum thickness of material to be used for lap joints in flat tank bottoms resting directly on grade or foundation shall be ⅜ in. (9.5 mm).

8.12.2 *Butt joints*. Butt joints may be used for welding all thicknesses of material permitted to be welded under the provisions of this standard.

8.12.3 *Plates.* The maximum thickness of plates, except structural components and base plates, permitted to be welded under this standard shall be 2 in. (51 mm). Structural components that are part of the primary container, primary support systems, or both may exceed 2 in. (51 mm) in thickness, provided that they meet the requirements of Sec. 2.2.3.1.1. Structural components that attach to the primary container to balance membrane discontinuities and base plates are excluded from these requirements. (See Table QW-422 of ASME BPVC Sec. IX, and Sec. 10.3 and Sec. 10.4 of this standard.)

Sec. 8.13 Lap Restrictions for Welded Lap Joints

Welded lap joints shall be lapped not less than five times the nominal thickness of the thinner plate joined ($5t$); however, the lap need not exceed 2 in. (51 mm) in double-welded lap joints, and need not exceed 1 in. (25 mm) in single-welded lap joints. The maximum lap of single-welded roof joints shall be 4 in. (102 mm), unless the joint is held together by intermittent fillet welds, continuous seal welds, or supported by a structural member.

Sec. 8.14 Minimum Size of Fillet and Seal Welds

8.14.1 *Fillet welds.* Plates 3⁄16 in. (4.76 mm) and less in thickness shall have fillet welds equal to the base metal thickness. Plates more than 3⁄16 in. (4.76 mm) thick shall have welds of a size not less than one-third the thickness of the thinner plate at the joint, with a minimum of 3⁄16 in. (4.76 mm).

8.14.1.1 Adjustment for root opening. Except for seal welds, the size of fillet welds shall be increased by the amount of root opening in excess of ¹⁄₁₆ in. (1.6 mm), and the root opening shall not exceed ³⁄₁₆ in. (4.76 mm).

8.14.2 *Seal welds.* Seal welding, when specified or required by this standard, shall be accomplished by a continuous weld combining the functions of sealing and strength with weld sections changed only as required strength may necessitate. Seal welds shall be of minimum size but sufficient to prevent cracking from thermal shrinkage. Seal weld acceptance shall be the same as for visual inspection of structural welds.

Sec. 8.15 Minimum Length of Welds

The minimum length of any weld shall be four times the weld size but not less than 1½ in. (38 mm).

8.15.1 *Fillet welds.* The effective length of a fillet weld shall not include the length of tapered ends. A deduction of at least ¼ in. (6.4 mm) shall be made from the overall length as an allowance for tapered ends.

Sec. 8.16 Intermittent Welding

Intermittent welding shall not be used on tank-shell or riser surfaces in contact with tank contents or on plate surfaces exposed to external weathering. Seal welds in accordance with Sec. 8.14.2 shall be used on these surfaces.

8.16.1 *Length.* The length of any segment of intermittent welds shall not be less than four times the weld size but never less than 1½ in. (38 mm).

8.16.2 *Seams.* Seams that are to have intermittent welds shall have continuous lengths of welds at each end for a distance of at least 6 in. (152 mm).

Sec. 8.17 Corrosion Protection

Welded joints in contact with stored water and exterior welded joints exposed to rain or rain runoff shall be seal welded.

Sec. 8.18 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 the subsequent cleaning and painting operations. Weld seams need not be chipped or ground, provided that they have a surface suitable for the subsequent cleaning and painting operations.

SECTION 9: SHOP FABRICATION

Sec. 9.1 Workmanship

Work performed on tanks built under the provisions of this standard shall be quality workmanship.

Sec. 9.2 Straightening

Any required straightening of material shall be done using methods that will not harm the steel. Minor cold straightening is permitted. Cold straightening may be performed by hammering or, preferably, by rolling or pressing. Heat may be used in straightening more severe deformations, unless otherwise specified. The steel temperature shall not exceed 1,200°F (649°C) for as-rolled and normalized steel, and 1,100°F (600°C) for quenched and tempered steel.

Sec. 9.3 Finish of Plate Edges—Welded Work

The plate edges to be welded may be universal mill edges or they may be prepared by shearing, machining, chipping, or by mechanically guided oxyfuel gas or plasma arc cutting. Edges of irregular contours may be prepared by manually guided oxyfuel gas or plasma arc cutting.

9.3.1 *Oxyfuel gas or plasma arc cutting.* When edges of plates are oxyfuel gas or plasma arc cut, the surface obtained shall be uniform and smooth and shall be cleaned of slag accumulation before welding. All cutting shall follow closely the lines prescribed.

9.3.2 *Shearing.* Shearing may be used for material ½ in. (13 mm) or less in thickness to be joined by butt joints and for all thicknesses of material permitted to be joined by lap joints. Burrs shall be removed.

Sec. 9.4 Rolling

Table 15 provides rolling requirements for shell plates for elevated tanks. Shell plates do not require rolling for tanks having a diameter larger than the minimum diameter indicated in Table 15 and for the plate thicknesses given therein.

Table 15 Minimum diameter for unrolled shell plates for elevated tanks

Plate Thickness				Minimum Diameter	
Minimum		Maximum			
in.	*(mm)*	*in.*	*(mm)*	*ft*	*(m)*
		<⅜	(<10)	40	(12.2)
>⅜	(>10)	½	(13)	60	(18.3)
>½	(>13)	⅝	(16)	120	(36.6)
>⅝	(>16)			Must be rolled for all diameters	

Sec. 9.5 Double-Curved Plates

Plates that are curved in two directions may be pressed either cold or hot or may be dished with a "mortar and pestle" die by repeated applications.

Sec. 9.6 Columns

9.6.1 *Milling of columns*. The ends of columns shall be milled to provide a satisfactory bearing unless the design calls for sufficient welding to resist the total calculated loads.

9.6.2 *Column fabrication tolerances*. The column axis shall not deviate from a straight line by more than 0.1 percent of the laterally unsupported length. At no cross section shall the difference between the maximum and minimum outside diameter of a tubular column exceed 2 percent of the nominal outside diameter. Local dents in tubular columns shall be no deeper than the thickness of the column shell.

Sec. 9.7 High-Strength Anchor Bolts

When high-strength anchor bolts are used, no welding, heating, or peening of the bolt is permitted.

Sec. 9.8 Shipping

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

SECTION 10: ERECTION

Sec. 10.1 Welds

All welds in the tank and structural attachments shall be made in a manner to ensure complete fusion with the base metal, within the limits specified for each joint, and in strict accordance with the qualified welding procedure specifications.

10.1.1 *Weather and temperature conditions*. Welding shall not be performed when the surfaces of the parts to be welded are wet from rain, snow, or ice; when rain or snow is falling on such surfaces; or during periods of high winds, unless the welder or welding operator and the work are properly protected. See Sec. 10.3 for preheat requirements and Sec. 10.4 for low-hydrogen requirements.

10.1.2 *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 with a blunt-nosed tool.

Table 16 Maximum thickness of reinforcement for butt joints

Plate Thickness	Maximum Thickness of Reinforcement			
	Vertical Joints		Horizontal Joints	
in. (*mm*)	*in.*	*(mm)*	*in.*	*(mm)*
≤½ (13)	3⁄32	(2.4)	⅛	(3.2)
>½ (13) ≤ 1 (25)	⅛	(3.2)	3⁄16	(4.8)
>1 (25)	3⁄16	(4.8)	¼	(6.4)

10.1.3 *Contour*. The surface beads shall merge smoothly into each other in all welds.

10.1.3.1 Undercut. Undercutting of base metal in the plate adjoining the weld shall be repaired, except as permitted in Sec. 11.4.2.1.

10.1.3.2 Craters. All craters shall be filled to the full cross section of the weld.

10.1.4 *Reinforcement.* The thickness of the reinforcement of the welds on all butt joints shall not exceed the thicknesses given in Table 16. The reinforcement need not be removed except to the extent that it exceeds the maximum acceptable thickness or when required for radiographic inspections. In no case shall the face of the weld lie below the surface of the plates being joined.

10.1.5 *Gouging.* Gouging at the root of welds and gouging of welds to remove defects may be performed with a round-nosed tool or by arc or oxygen gouging.

10.1.6 *Cleaning between beads.* Each bead of a multiple-pass weld shall be cleaned of slag and other loose deposits before the next bead is applied.

Sec. 10.2 Preparation of Surfaces to Be Welded

Except as otherwise provided in the following, surfaces to be welded shall be free from loose scale, slag, heavy rust, grease, oil, paint, and any other foreign material except tightly adherent mill scale. A light film of deoxyaluminate or equivalent splatter film compound may be disregarded. Such surfaces shall also be smooth, uniform, and free of fins, tears, and other defects that adversely affect proper welding. A fine film of rust adhering on cut or sheared edges after wire brushing need not be removed.

10.2.1 *Protective coatings.* If protective coatings, deoxyaluminate, or equivalent are to be used on surfaces to be welded, the protective coating shall be included in welding procedure qualification tests for the brand formulation and maximum thickness of coating to be applied; except, if thickness of coating does not exceed 2 mils (51 μm), then qualifications with or without coating are acceptable.

Sec. 10.3 Preheating Weld Joints

10.3.1 *Preheat.* When preheating is required, the base metal within a distance of four times the plate thickness (3 in. [76 mm] minimum, but need not exceed 6 in. [152 mm]) from the location where welding is started shall be preheated to at least the preheat temperature specified in Sec. 10.3.2. That temperature shall be maintained for a distance of four times the plate thickness (3 in. [76 mm] minimum but need not exceed 6 in. [152 mm]) ahead of the arc as welding progresses, unless otherwise specified. The preheat temperature shall be the more restrictive of the preheat temperature given in this section and the preheat temperature given in the qualified welding procedure specification. For multipass welds, the preheat requirements shall apply to each pass.

10.3.2 *Preheat requirements.*

10.3.2.1 Plate thicknesses less than or equal to 1½ in. (38 mm). Preheat requirements for plate thicknesses less than or equal to 1½ in. (38 mm):

1. For base-metal temperatures greater than or equal to 32°F (0°C), no preheat is required.

2. For base-metal temperatures greater than or equal to 0°F (–18°C) and less than 32°F (0°C), the minimum preheat temperature is 100°F (38°C).

3. For base-metal temperatures less than 0°F (–18°C), the minimum preheat temperature is 200°F (93°C). In addition, the base metal along the length of the weld joint in the direction of welding shall be preheated and maintained as welding progresses for a distance of at least 36 in. (914 mm) from the point of welding or the entire weld joint length, whichever is less. See Sec. 10.4 for low-hydrogen requirements.

10.3.2.2 Plate thicknesses greater than 1½ in. (38 mm). Preheat requirements for plate thicknesses greater than 1½ in. (38 mm):

1. For base-metal temperatures greater than or equal to 70°F (21°C), no preheat is required.

2. For base-metal temperatures greater than 32°F (0°C) and less than 70°F (21°C), no preheat is required when low-hydrogen electrodes or low-hydrogen welding processes are used. Otherwise, the minimum preheat base-metal temperature is 200°F (93°C).

3. For base-metal temperatures less than or equal to 32°F (0°C), the minimum preheat temperature is 200°F (93°C).

4. For base-metal temperatures less than 0°F (–18°C), the base metal along the length of the weld joint in the direction of welding shall be preheated and

maintained as welding progresses for a distance of at least 36 in. (914 mm) from the point of welding or the entire weld joint length, whichever is less. See Sec. 10.4 for low-hydrogen requirements.

Sec. 10.4 Low-Hydrogen Electrodes and Welding Processes

10.4 1 *Handling.* After low-hydrogen filler metal has been removed from its original package, it shall be protected or stored so that its characteristics and welding properties are not affected. Low-hydrogen electrodes shall be maintained in accordance with electrode conditioning recommendations contained in AWS A5.1/A5.1M or A5.5/A5.5M, whichever is applicable.

10.4.2 *Low temperatures.* If welding is to be performed when the base metal temperature is lower than 0°F (–18°C), low-hydrogen electrodes or low-hydrogen welding processes shall be used.

Sec. 10.5 Tack Welds

Tack welds used in the assembly of joints subject to primary stress from the weight or pressure of tank contents shall be made by qualified welders and shall be thoroughly cleaned of all welding slag, but need not be removed, provided that they are visually inspected for soundness (no cracks, complete fusion, filled craters, and acceptable profiles) and are thoroughly fused into the subsequently deposited weld metal. Tack welds used in the assembly of joints subject to secondary stress, such as those used in flat bottoms, roofs, and circumferential seams of cylindrical tank shells, need not be removed, provided that they are sound, cleaned of all welding slag, and that the subsequently applied weld beads are thoroughly fused into the tack welds.

Sec. 10.6 Tank Assembly

Shell, bottom, and roof plates subjected to stress by the weight or pressure of the contained liquid shall be assembled and welded in such a manner that the proper curvature of the plates in both directions is maintained.

10.6.1 *Clips, jigs, and lugs.* Any clips, jigs, or lugs welded to the shell plates for erection purposes shall be removed without damaging the plates, and any portion of weld beads remaining shall be chipped or ground smooth.

10.6.2 *Bottom plates for elevated tanks.* The bottom plates for elevated tanks shall be assembled and welded together by a procedure that will result in a minimum of distortion from weld shrinkage.

10.6.3 *Bottom plates for flat-bottom tanks.* The bottom plates for flat-bottom tanks, after being laid out and tacked, shall be joined by welding the joints in a sequence that results in the least distortion caused by shrinkage of the weld. Out-of-plane distortion equal to 1 percent of the tank radius is considered acceptable.

10.6.4 *Tank shell.* For welding in the vertical position, the progression of welding shall be either upward or downward, according to the direction specified in the welding procedure and used for welder-performance qualification. The shell plates shall be joined by welding the joints in a sequence that results in the least distortion caused by shrinkage of the weld and that will avoid kinks at the longitudinal joints.

10.6.5 *Ground-supported standpipe and reservoir cylindrical shell tolerances.* Shell tolerances may be waived if the structural adequacy of the shell is substantiated by a rational analysis.

10.6.5.1 Plumbness. The maximum out-of-plumbness of the top of the shell relative to the bottom of the shell shall not exceed 1/200 of the total shell height.

The out-of-plumbness in one shell plate shall not exceed the permissible variations for flatness and waviness as specified in ASTM A6/A6M or A20/A20M, whichever is applicable.

10.6.5.2 Roundness. Radii measured at 1 ft (0.3 m) above the bottom corner weld shall not exceed the tolerances as given in Table 17.

10.6.5.3 Peaking and banding at weld joints. Peaking is the out-of-plane distortion across a vertical weld seam. Banding is the out-of-plane distortion across a circumferential weld seam.

1. Using a horizontal sweep board 36 in. (0.91 m) long, peaking shall not exceed ½ in. (13 mm).
2. Using a vertical sweep board 36 in. (0.91 m) long, banding shall not exceed ½ in. (13 mm).

Table 17 Roundness—cylindrical shells

Diameter Max		**Radius Tolerance**	
ft	***(m)***	***in.***	***(mm)***
40	(12.2)	±½	(±13)
150	(45.7)	±¾	(±19)
<250	(<76.2)	±1	(±25)
≥250	(≥76.2)	±1¼	(±32)

10.6.5.4 Localized flat spots. Flat spots (that deviate from the theoretical shape) measured in the vertical plane shall not exceed the appropriate plate flatness and waviness requirements in ASTM A6/A6M and A20/A20M, whichever is applicable.

10.6.6 *Erection tolerances for allowable compressive stress formulas.* Double-curved, axisymmetrical, conical, and cylindrical sections subject to the provisions of Sec. 3.4.3 shall be assembled and welded in such a manner that the following tolerances are obtained:

10.6.6.1 Local deviation from theoretical shape: Except as noted, the maximum local deviation from theoretical shape, e_x, is defined by the following equations:

$$e_x = 0.01L_x \tag{10-1}$$

$$L_x = 4\sqrt{Rt} \tag{10-2}$$

Where:

L_x = gauge length to measure local imperfection

e_x = local deviation from theoretical shape

t = shell thickness

R = radius of exterior surface of the shell, normal to the plate at the point under consideration and measured from the exterior surface of the plate to the axis of revolution

NOTE: All units must be consistent.

For ground-supported flat-bottom tanks, when the ratio of the calculated compressive stress to the allowable compressive stress, C_{1cr}, is less than 0.75, the erection tolerance defined by Eq 10-1 may be increased by multiplying by the factor K_{1cr} as determined by Eqs 10-3 through 10-5. The erection tolerance shall be established for each element and shall be based on the load combination that produces the smallest factor, K_{1cr}.

For $C_{1cr} \geq 0.75$,

$$K_{1cr} = 1.0 \tag{10-3}$$

For $0.75 > C_{1cr} \geq 0.25$,

$$K_{1cr} = 1.0 + \frac{0.75 - C_{1cr}}{0.50} \tag{10-4}$$

For $C_{1cr} < 0.25$, $$K_{1cr} = 2.0 \tag{10-5}$$

Where:

C_{1cr} = ratio of actual compressive stress to allowable compressive stress

K_{1cr} = allowable local deviation multiplier

10.6.6.2 Offset of aligned courses. Alignment shall comply with Sec. 10.7.2.

Sec. 10.7 Matching Plates

10.7.1 *Lap joints.* The plates forming a lap joint shall be held in as close contact as possible during welding. Where plate separation occurs, the size of weld shall be increased by the amount of the separation (see Sec. 8.14.1.1).

10.7.2 *Butt joints.* The adjoining edges of butt joints shall be aligned accurately and retained in position during welding, so that the offset tolerances of Table 18 are not exceeded in the welded joint.

Sec. 10.8 Grouting of Column, Riser, and Single-Pedestal Bases for Elevated Tanks

After the tank has been completely erected and "trued up," a minimum 1-in. (25-mm) space between column, riser, and single-pedestal bases and the foundation shall be provided for grouting. The space shall be cleaned, thoroughly wetted, and filled with a 1:1.5 cement–sand grout or commercial grout. The grout shall be forced under the bases until the space is filled completely.

Sec. 10.9 High-Strength Anchor Bolts

When high-strength anchor bolts are used, no welding, heating, or peening of the anchor bolt is permitted. High-strength anchor bolts shall be pretensioned to at least 80 percent of the design load.

Table 18 Maximum allowable offset of aligned butt joints

Thickness		Subject to Primary Stress		Subject to Secondary Stress	
in.	***(mm)***	***in.***	***(mm)***	***in.***	***(mm)***
$0 < t^* \leq ⅝$	$0 < t \leq 16$	1⁄16	1.6	⅛	3.2
$t > ⅝$	$t > 16$	Lesser of $0.10t$ or ¼	Lesser of $0.10t$ or 6	Lesser of $0.20t$ or ⅜	Lesser of $0.20t$ or 9.5

* t = nominal thickness of the thinner plate at the joint.

SECTION 11: INSPECTION AND TESTING

Sec. 11.1 General

Work, whether performed in the shop or field, shall be inspected in accordance with this standard.

Sec. 11.2 Inspection Report

A certified copy of a written report confirming that the work was inspected as set forth herein shall be provided when specified. The report shall include the following:

1. A copy of welder performance qualifications.
2. A summary of inspection of radiographs and inspection by air carbon arc gouging, if used.
3. Identification of unacceptable radiographs and inspections by arc gouging and a statement of the action taken to rectify unsatisfactory welds.
4. Record of welders employed on each joint, if applicable (see Sec. 8.3).
5. Certified record of welders (see Sec. 8.2.2.1).

Radiographs and inspection records shall be provided when specified.

Sec. 11.3 Welders' Credentials

All welders or welding operators shall have current qualification records, or they shall be tested before any welding is performed (refer to Sec. 8.2).

Sec. 11.4 Inspection of Welded Joints

11.4.1 *Radiographic inspection.* Inspection of welded joints by radiographic testing, as described in Sec. 11.6, shall be confined to joints of the tank cylindrical shell, tank bottom, roof knuckles, and tank roof, particularly those subject to primary stress from weight or pressure of tank contents and load-bearing risers in contact with the water, except as exempted in Sec. 11.4.1.2. For inspection purposes, primary tensile stress shall be considered a primary stress, and primary compression stress shall be considered a secondary stress in Sec. 11.4 and Sec. 11.5.

11.4.1.1 Inspection method. Inspection of complete joint penetration welded-shell butt joints and load-bearing risers in contact with water shall be made by the radiographic method in Sec. 11.6. Primary stress joints that cannot be radiographed may be inspected by air carbon arc gouging as described in Sec. 11.8.

11.4.1.2 Limitations of radiographic inspection. Radiographic inspection shall apply to complete penetration welded butt joints only. Inspection by radiographic methods is not required for butt joints for which partial penetration welds are allowed (Sec. 8.10 and Sec. 8.4.2) and the following joints:

1. Welds in roof plates and roof knuckles where the required thickness due to hydrostatic pressure is less than 25 percent of the as-ordered thickness.

2. Welds in flat tank bottoms resting directly on grade or foundation other than annular plates required by Sec. 14.3.2.9.

3. Welds joining flat tank bottoms to the first rings of the tank shell.

4. Splice welds in the top angle and welds connecting the top angle to the shell or roof.

5. Welds connecting manholes to the tank.

6. Welds connecting appurtenances to the tank.

7. Any other fillet welds not previously included.

11.4.2 *Visual inspection*. All welds, except as noted in Sec. 11.4.4.2, 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, except as noted in Sec. 11.4.4.

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 allowable limits given in Sec. 10.1.4 and Table 16.

7. Fillet weld convexity in excess of

a. $\frac{1}{16}$ in. (1.6 mm) for width of individual bead or weld face not over $\frac{5}{16}$ in. (7.9 mm).

b. $\frac{1}{8}$ in. (3.2 mm) for width of individual bead or weld face over $\frac{5}{16}$ in. (7.9 mm), but less than 1 in. (25.4 mm).

c. $\frac{3}{16}$ 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. 11.4.2.1.

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 the limits given in Sec. 10.7 and Table 18.

11.4.2.1 Maximum permissible undercut.

11.4.2.1.1 Undercut is a groove melted into the base metal adjacent to the weld toe or weld root that is left unfilled by weld metal.

11.4.2.1.2 For butt joints subject to primary stress due to weight or pressure of tank contents, maximum permissible undercut shall be 1⁄64 in. (0.4 mm).

11.4.2.1.3 For butt joints subject to secondary stress, penetration is required only within the limits established in Sec. 8.4.2. A maximum undercut of 1⁄32 in. (0.8 mm) is permitted, provided that the unwelded portion plus the undercut shall not reduce the thickness of the joint by more than one-third of the thickness of the thinner plate joined.

11.4.2.1.4 For lap joints subject to primary stress due to weight or pressure of tank contents, the maximum permissible undercut shall be 1⁄64 in. (0.4 mm).

11.4.2.1.5 For lap joints subject to secondary stress, the maximum undercut permitted shall be 1⁄32 in. (0.8 mm).

11.4.2.1.6 For butt joints in tension bracing, the finished surface of the splice weld shall be ground to eliminate sharp notches, and all undercut shall be repaired to provide a smooth transition from the splice weld to the brace material (i.e., no undercut).

11.4.2.1.7 The maximum undercut permitted for all other joints shall be 1⁄32 in. (0.8 mm).

11.4.3 *Measurement and documentation of shell imperfections.*

11.4.3.1 Ground-supported standpipes and reservoirs. A visual inspection of cylindrical shells, supplemented by measurements, shall be used to verify compliance with the erection tolerances of Sec. 10.6.5 and Table 17.

11.4.3.2 Sections governed by Sec. 3.4.3. Double-curved axisymmetrical, conical, and cylindrical sections governed by Sec. 3.4.3 local buckling allowable stresses shall be inspected as follows:

11.4.3.2.1 A visual inspection of shells designed using Method 1 (see Sec. 3.4.3.1), supplemented by measurements, shall be used to verify compliance with the tolerances of Sec. 10.6 and Sec. 10.7.

11.4.3.2.2 Shells designed using Method 2 (see Sec. 3.4.3.2) shall be verified and documented as follows: Measurements shall be obtained to establish the as-constructed meridional profile of each shell plate. The profile shall be established at and midway between each meridional seam. Additional measurements shall be taken when the shell profile appears to be irregular. If measurements indicate that the shell does not comply with the tolerances of Sec. 10.6.6, additional measurements shall be taken to determine the extent of the non-conforming area. Documentation of field measurements and a statement certifying compliance with the tolerances of Sec. 10.6.6 and Sec. 10.7 shall be provided. Where the tolerances of Sec. 10.6.6 and Sec. 10.7 are not met, further evaluation is required and corrective action may be required (see Sec. 3.4.3).

11.4.4 *Support columns for elevated tanks*. Except as noted, this section applies to tubular support columns of multiple-column tanks, large-diameter dry risers, and single-pedestal columns with either a smooth cylindrical, conical, or bent plate surface.

11.4.4.1 Circumferential butt joints. This section only applies to large-diameter dry risers and single-pedestal columns. Welds made from one side into backup bars or similar one-side welding may show dark lines intermittently on a radiograph at the root of the joint. This is normal and is not cause for rejection, provided that the welds are otherwise considered acceptable. For one-sided welds, some nonfusion or lack of penetration less than 10 times the column wall thickness in length and rounded indications shall not be cause for rejection, provided that the governing stress is compression.

11.4.4.2 Longitudinal butt joints. Longitudinal butt joints shall be visually inspected on the total outside length and for a distance of one diameter on the inside of each section welded. Areas less than 10 times the column wall thickness in length, with lack of penetration not exceeding 1⁄32 in. (0.8 mm) deep, are acceptable, and the requirements of Sec. 11.4.2.1.3 shall apply for exterior welds.

11.4.4.3 Lap joints. For lap joints, the requirements of Sec. 11.4.2.1.5 apply for both interior and exterior welds.

11.4.5 *Tension-member bracing*. Splice joints shall be inspected by radiography, ultrasonic testing, or by proof test to 2⁄3 times the published minimum yield strength. For each welder of this joint, testing shall be as follows:

11.4.5.1 Qualification testing. Before production welding, each welder shall be qualified by successfully completing an initial test of a welded splice joint.

Testing shall be in accordance with Sec. 11.4.5. The test qualifies the welder for the qualification rod diameter and all smaller rod diameters.

11.4.5.2 Production testing. For each welder, at least one splice joint shall be tested for every 300 splice joints welded, with a minimum of one test every 16 weeks. The test rod diameter shall be no smaller than two-thirds of the qualification rod diameter in Sec. 11.4.5.1.

Sec. 11.5 Number and Location of Radiographs for Butt Weld Joints in Tank Shells, Load-Bearing Risers, and Single-Pedestal Columns

Inspection shall be performed as the work progresses but, including the shot in the first 10 ft (3 m) for a welder, need not be performed until after all the joints accessible from one scaffold position have been welded. Refer to Sec. 11.6 for radiographic testing procedures.

11.5.1 *Joints of the same type and thickness.* Tank shell and wet riser joints of the same type and thickness, based on the thickness of the thinner plate at the joint, which are subject to primary stress due to weight or pressure of tank contents, shall have one radiograph taken in the first 10 ft (3 m) of completed joint welded by each welder or welding operator. Thereafter, without regard to the number of welders or welding operators working thereon, one additional radiograph shall be taken in each additional 100 ft (30 m) and any remaining major fraction thereof. The radiograph locations selected for seams subject to primary stress shall include 5 percent of all junctures of joints that include at least one seam subject to primary stress, with a minimum of two such junctures per tank.

11.5.2 *Tank shell and wet riser—secondary stress.* Tank shell and wet riser joints of the same type and thickness, based on the thickness of the thinner plate at the joint, which are subject to secondary stress, shall have one radiograph taken in the first 10 ft (3 m) of completed joint without regard to the number of welders or welding operators working thereon. Thereafter, one additional radiograph shall be taken in each additional 200 ft (60 m) and any remaining major fraction thereof. When portions of the joints have partial joint penetration welds, the location of the radiographs shall be selected from that portion of the seam containing the complete joint penetration welds, per Sec. 8.4.2.

11.5.3 *Plate thickness.* For the purposes of Sec. 11.5.1 and Sec. 11.5.2, plates shall be considered to be of the same thickness when the difference in the specified thickness does not exceed ⅛ in. (3 mm). Radiographic film coverage for each thickness shall conform to Sec. 11.6.6.

11.5.4 *Radiographs for multiple tanks.* When two or more tanks are erected, either concurrently or continuously, in the same location, the number of radiographs to be taken (Sec. 11.5.1 and Sec. 11.5.2) may be based on the aggregate footage of welds of the same type and thickness in such group of tanks, rather than on the footage in each individual tank.

11.5.5 *Multiple welders on single joint.* It is to be recognized that the same welder or welding operator may or may not weld both sides of the same butt joint. Therefore, it is permissible to test two welders' or welding operators' work with one radiograph. When an inspection of this type is rejected, it must be determined whether one or both welders or welding operators were at fault through examination of the radiograph or by subsequent tests of each welder's or welding operator's work. Insofar as possible, an equal number of locations shall be examined from the work of each welder or welding operator on the tank, except that this requirement shall not apply where the length of seams welded by a welder or welding operator is much less than the average.

11.5.6 *Single-pedestal columns and large-diameter dry risers.* Single-pedestal columns and large-diameter dry risers more than 36 in. in diameter not in contact with the water shall have one radiograph taken in the first 10 ft (3 m) of completed circumferential butt weld joint without regard to the number of welders or welding operators. Thereafter, without regard to the number of welders or welding operators, one spot radiograph shall be examined in each additional 200 ft (60 m) and any remaining major fraction thereof. No spot radiograph need be taken at junctures of circumferential and longitudinal joints.

Sec. 11.6 Procedures for Inspection of Welded-Shell Butt Joints—Radiographic Testing

The inspection of welded-shell butt joints for which complete joint penetration is specified shall be made by x-ray or gamma-ray methods. No credit on the value for joint efficiency shall be allowed for such radiographic inspection.

11.6.1 *Application.* The procedure outlined shall apply only to complete penetration welded-shell butt joints.

11.6.2 *Radiographic examination method.* Except as modified in this section, the radiographic examination method shall be in accordance with ASME BPVC Sec. V, Article 2.

11.6.3 *Level II radiographers.* Level II radiographers shall perform the final acceptance of the radiographs. Level II radiographers shall be qualified in accordance with the current edition of ASNT SNT-TC-1A and all supplements, NDT Level II.

Table 19 Maximum height of weld reinforcement of weld for butt joints above plate surface

Plate Thickness				Maximum Height of Crown	
Minimum		Maximum			
in.	*(mm)*	*in.*	*(mm)*	*in.*	*(mm)*
		½	(13)	⅟16	(1.6)
>½	(>13)	1	(25)	3⁄32	(2.4)
>1	(>25)			⅛	(3.2)

11.6.4 *Final acceptance of radiographs.* The requirements of Sec. T-274 and Sec. T-285 of ASME BPVC Sec. V are to be used only as a guide. Final acceptance of radiographs shall be based on the ability to see the prescribed penetrometer image and the specified essential hole or the essential wire.

11.6.5 *Finished reinforcement surface.* The finished surface of reinforcement at the location of the radiograph may be flush with the plate or may have a reasonably uniform crown not to exceed the values in Table 19.

11.6.6 *Radiographic film.* Each radiograph shall clearly show a minimum of 6 in. (152 mm) of weld length except for a junction of vertical and horizontal welds, which shall clearly show not less than 2 in. (50 mm) of horizontal weld length on each side of the vertical intersection and a minimum of 3 in. (75 mm) of weld length on the vertical seam. The film shall be centered on the weld and shall be of sufficient width to permit adequate space for the location of identification markers and a thickness gauge or penetrometer.

11.6.7 *Radiographic procedure*. The weld shall be radiographed by a technique that will determine quantitatively the size of defects in accordance with the sensitivity required by Table T-276 of ASME BPVC Sec. V, based on the average thickness of the two plates joined plus weld reinforcement.

11.6.8 *Penetrometer placement.* One penetrometer shall be used for each film, to be placed adjacent to or across the weld seam at the approximate center of the location to be examined. For vertical welds, the penetrometer shall be placed parallel to the seam; for horizontal welds, the penetrometer shall be placed parallel to the weld seam. Wire penetrometers shall be placed across the weld. See Sec. T-277 of ASME BPVC Sec. V for more details.

11.6.9 *Review of radiographs.* Radiographs shall be reviewed before any repairs of welds.

11.6.10 *Radiographic standards.* Except as permitted in Sec. 11.4.4, sections of welds shown by radiography, in addition to visual inspection, to have any of the following imperfections shall be judged unacceptable:

1. Any crack, incomplete fusion, or inadequate penetration.
2. Any individual elongated inclusion having a length greater than two-thirds the thickness of the thinner plate of the joint except that, regardless of the plate thickness, no such inclusion shall be longer than ¾ in. (19 mm), and no such inclusion shorter than ¼ in. (6 mm) shall be cause for rejection.
3. Any group of inclusions in line, in which the sum of the longest dimensions of all such imperfections is greater than T (where T is the thickness of the thinner plate joined) in a length of $6T$, except when the space between every pair of adjacent imperfections is greater than three times the length of the longer of the imperfections; when the length of the radiograph is less than $6T$, the permissible sum of the lengths of all inclusions shall be proportionately less than T, provided that the limits of the deficient welding are clearly defined.
4. Rounded indications in excess of those shown as acceptable in ASME BPVC Sec. VIII, Div. 1, Appendix 4.

11.6.11 *Defective welds.* When a section of weld is shown by a radiograph to be unacceptable or the limits of the deficient welding are not defined by such radiograph, two adjacent radiographs shall be taken. However, if the original radiograph shows at least 3 in. (76 mm) of acceptable weld between the defect and any one edge of the film, an additional radiograph need not be taken on that side of the defect. If the weld at the first adjacent radiograph fails to comply with the requirements of Sec. 11.6.10, additional adjacent radiographs shall be made until the limits of unacceptable welding are determined. Alternatively, if the defect extends beyond the first adjacent radiographs, the complete defect may be determined by air carbon arc gouging. A final radiograph shall be taken at the end of the air carbon arc gouge to ensure that the entire defect has been removed. Welding performed by the welder or welding operator on that joint may be replaced, in which case one radiograph shall be taken at any selected location on any other joint on which the same welder or welding operator has welded. If any of the additional radiographs fail to comply with the requirements of Sec. 11.6.10, the limits of unacceptable welding shall be determined as previously described.

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

Sec. 11.7 Procedure for Inspection of Groove Welds in Tension Member Bracing by Ultrasonic Inspection

11.7.1 *Ultrasonic testing (UT).* Personnel performing the examinations shall be qualified in accordance with ASNT SNT-TC-1A. For a full ultrasonic inspection, each groove weld shall be straight-beam inspected circumferentially around the entire weld and shall be angle-beam inspected axially in both directions circumferentially around the entire weld. Level II personnel shall perform the final acceptance.

1. The weld groove shall be of a configuration that ensures full ultrasonic coverage.
2. Ultrasonic technique details shall be as outlined in ASME BPVC Sec. V, Article 5.
3. Ultrasonic acceptance standards shall be as shown in ASME BPVC Sec. VIII, Div. 1, Appendix 12, paragraph 12-3.

Sec. 11.8 Inspection by Air Carbon Arc Gouging

In those areas where radiographic inspection is not feasible, an inspection of welds by an experienced inspector may be made by air carbon arc gouging. A form shall be prepared identifying the joint, justification for this type of inspection, length of inspection, and results of inspection.

11.8.1 *Testing procedure*. The number of test sections shall be determined by Sec. 11.5. A portion of the weld, approximately 2 in. (50 mm) long, shall be gouged out to the root of the weld. Visual inspection shall be made for sound welding, lack of penetration or fusion, cracks, or porosity. If unacceptable defects are found, additional areas shall be gouged to isolate the undesirable area.

11.8.2 *Repair procedure*. All gouged areas shall be repair welded using a procedure that will produce a weld to its specified size, contour, and quality.

Sec. 11.9 Repair of Defective Welds

Defective welds shall be removed by grinding, chipping with a round-nosed tool, or by air arc or oxygen gouging, from one or both sides of the joint, and then rewelded in compliance with approved procedures. Removal of defective welds is required only to the extent necessary to remove the defects present. Repairs shall be reinspected by the original test procedure.

Sec. 11.10 Testing

11.10.1 *Flat bottoms*. On completion of welding of the tank bottom and before painting, the tank bottom shall be tested for water tightness by one of the following methods.

11.10.1.1 Magnetic-particle testing. The joints may be tested by the magnetic-particle method.

11.10.1.2 Air-pressure or vacuum testing. Air pressure or vacuum may be applied to the joint, using soapsuds, linseed oil, or other suitable material for the detection of leaks. The gauge should register a vacuum of at least 2 psi (13.8 kPa).

11.10.2 *Shell-to-bottom joint*. Prior to painting, the inside fillet weld shall be tested for leaks by one of the following methods:

1. Test the inside fillet weld with penetrating oil before welding the outside fillet weld.
2. Vacuum box test the inside fillet weld at 2 psig (4.1 in. Hg) (13.8 kPa gauge) to 4 psig (8.2 in. Hg) (27.6 kPa gauge) either before or after welding the outside fillet weld.

11.10.3 *Shell, bottom, and roof*. Unless otherwise specified, the tank shall be hydrotested after painting by filling the tank with water to the MWL. Any leaks in the shell, bottom, or roof (if the roof contains water) shall be repaired by chipping, gouging, or oxygen gouging to remove any defective welds, and rewelded. No repair work shall be done on any joints unless the water in the tank is at least 2 ft (0.6 m) below the point being repaired.

SECTION 12: FOUNDATION DESIGN

Sec. 12.1 General Requirements

Construction drawings of the foundation shall be provided and shall include dimensions, loadings used in the design, design and construction standards used, materials of construction, and allowable soil pressure or deep foundation capacity. The type of foundation and foundation depth shall be based on a geotechnical investigation. 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.

12.1.1 *Water load*. Water load, as defined in Sec. 3.1.3, shall be considered as live load, as defined by ACI 318 (see Sec. 12.8). The appropriate factors for all live loads shall be used in the foundation design.

12.1.2 *Design snow load*. Unless otherwise specified, the design snow load, if any, need not be combined with wind or seismic soil-bearing pressures for design of footings, slabs, or piers.

Sec. 12.2 Soil-Bearing Value

The design soil-bearing pressure shall be specified and shall include an appropriate factor of safety (Sec. 12.3) that is based on a geotechnical investigation and the method of pile capacity testing, if applicable. In no case shall the specified bearing pressure cause settlements that may impair the structural integrity of the tank.

12.2.1 *Geotechnical investigation*. A geotechnical investigation shall be performed to determine the following:

1. The presence or absence of rock, old excavation, or fill.
2. Whether the site is suitable for the structure to be built thereon, and what remediation, if any, is necessary to make it suitable.
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 depth at which foundation must be founded.
7. Whether a deep foundation will be required and the type, capacity, and required length of piles, caissons, piers, etc.
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 estimated magnitude of uniform and differential settlement.
10. For standpipes and reservoirs, the minimum allowable foundation width for continuous and isolated footings, if applicable.
11. Site Class in accordance with Sec. 13.2.3.

Sec. 12.3 Safety Factors

The following safety factors shall be used as a minimum in determining the design soil-bearing pressure. The ultimate bearing capacity shall be based on sound principles of geotechnical engineering in conjunction with a geotechnical investigation.

Table 20 Minimum safety factor for design bearing capacity

Foundation Type and Method of Establishing Capacity	Load Combinations Without Wind or Seismic	Load Combinations with Wind or Seismic
Shallow foundations:		
Analysis using engineering principles	3.0	2.25
Deep foundations:		
Analysis using engineering principles	3.0	2.25
High-strain dynamic testing of piles in accordance with ASTM D4945	2.25	1.69
Static load testing in accordance with ASTM D1143 or high-strain dynamic testing of piles in accordance with ASTM D4945 using signal matching analysis or other in-situ load tests that determine end bearing, side friction, or both	2.0	1.5

Load combinations in Table 20 shall exclude overturning toe pressure caused by wind or seismic shear at the top of the footing, unless otherwise specified. A safety factor of 2.0 shall be provided based on calculated ultimate bearing capacity for gravity loads plus wind or seismic load, including overturning toe pressure caused by shear at the top of footing.

Sec. 12.4 Foundations for Cross-Braced Multicolumn Tanks

12.4.1 *Riser foundation.* The riser foundation shall accommodate the specified piping. The specified design soil-bearing values shall be such that differential settlement between the riser foundation and outer piers is minimized. The specified design soil-bearing values shall not be exceeded when the portion of dead and water loads carried by the riser, net weight (44 lb/ft^3 [705 kg/m^3]) of the concrete in the pier below the original ground line, and total weight of concrete and soil above the original ground line are included.

12.4.2 *Column foundations.* Column foundations may be of any suitable shape and shall be reinforced concrete. The weight of the pier plus the weight of the soil directly above the base of the pier or the tension allowable on a pile footing shall be sufficient to resist the maximum net uplift for tank-empty plus wind load or tank-full plus seismic load cases.

12.4.3 *Pier.* The size of the pier shall be such that the specified design soil-bearing value will not be exceeded when the following loads are included: net weight (44 lb/ft^3 [705 kg/m^3)] of concrete foundation below original ground line, full weight of concrete and soil above the original ground line, and portion of dead, water, and snow loads carried by the column pier. Maximum wind or seismic loads shall be combined with gravity loads, in accordance with Sec. 12.3. Peak toe pressure caused by shear at the top of the foundation shall be combined with gravity loads and wind or seismic loads, in accordance with Sec. 12.3.

12.4.4 *Batter.* For battered columns without bottom struts, the axis of column foundations shall have the same batter as the column. For battered columns with bottom struts attached to columns or with piers tied together and for vertical columns, the axis of the foundations shall be vertical.

12.4.5 *Size of top.* The tops of foundations shall project at least 3 in. (76 mm) beyond the column or riser base plates. The top corners shall be either neatly rounded or finished with suitable bevel.

12.4.6 *Tolerances on concrete foundation.* Tops of pedestals shall be troweled level to within ±¼ in. (±6 mm) of the theoretical elevation. Plan dimensions shall not be more than ½ in. (13 mm) less than specified dimensions. Centerline location of pedestals shall not vary more than ±½ in. (±13 mm) from the theoretical location.

12.4.7 *Tolerances on anchor bolt installation.* Unless otherwise specified, design of anchor bolts and anchor bolt attachments shall accommodate, and installation of anchor bolts shall comply with, the following tolerances:

1. Location in plan shall be no more than ¼ in. (6 mm) from the specified location.
2. Projection above top of concrete shall be ±¼ in. (±6 mm) from the specified projection.
3. Vertical misalignment shall not exceed ⅛ in. (3 mm) in 12 in. (305 mm).

Sec. 12.5 Foundations for Single-Pedestal Tanks

Single-pedestal tank foundations may consist of a reinforced concrete slab or ringwall footing. The size shall be such that the specified design soil-bearing value will not be exceeded when the following loads are included: net weight (44 lb/ft^3 [705 kg/m^3]) of concrete foundation below original ground line, full weight of concrete and soil above original ground line, deadweight of the structure, water load, and design snow load. Wind or seismic loads shall be combined with gravity loads, in accordance with Sec. 12.3.

12.5.1 *Overturning stability.* The size of the foundation shall be sufficient to maintain bearing pressures below the ultimate bearing capacity of the soil when

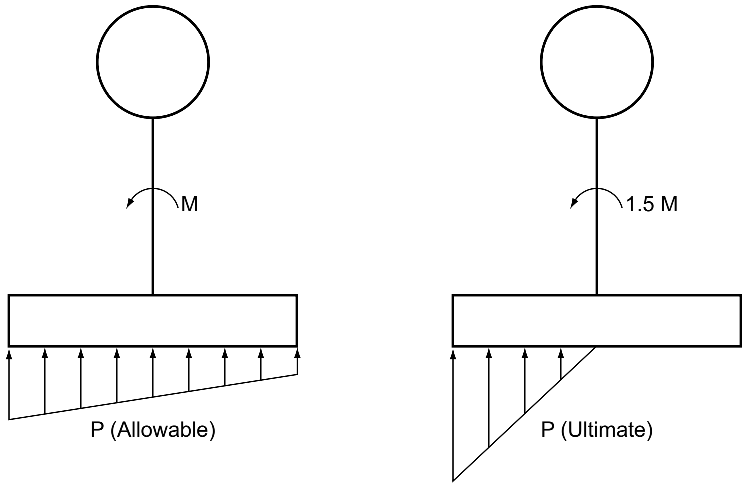


Figure 1 Diagram for checking overturning stability of pedestal-type elevated tanks (wind or seismic events)

subjected to an overturning moment equal to 1.5 times the overturning moment determined for wind or seismic loads (see Figure 1).

12.5.2 *Tolerances on concrete foundations.* Tops of ringwall footings shall be troweled level to within ±¼ in. (±6 mm) of the theoretical elevation. Plan dimensions shall not be more than ½ in. (13 mm) less than specified dimensions. Centerline location of ringwall footings shall not vary more than ±½ in. (±13 mm) from theoretical location.

12.5.3 *Tolerances on anchor bolt installation.* Unless otherwise specified, design of anchor bolts and anchor bolt attachments shall accommodate, and installation of anchor bolts shall comply with, the following tolerances:

1. Location in plan shall be no more than ¼ in. (6 mm) from the specified location.
2. Projection above top of concrete shall be ±¼ in. (±6 mm) from the specified projection.
3. Vertical misalignment shall not exceed ⅛ in. (3 mm) in 12 in. (305 mm).

Sec. 12.6 Foundations for Ground-Supported Flat-Bottom Tanks

Foundations for ground-supported flat-bottom tanks shall be one of the foundation types described in Sec. 12.6.1. The type of foundation shall be specified. Excavation, soil preparation, and compaction shall conform to accepted engineering practice for the bearing pressures predicted; refer also to Sec. 12.9. Site grading around the tank shall provide for positive drainage away from the tank. The top of the foundation shall be a minimum of 6 in. (152 mm) above the finished grade, unless otherwise specified. Unless otherwise specified, the foundation shall be graded to slope uniformly upward to the center of the tank with a minimum slope of 1 in. (25 mm) vertical to 10 ft (3.0 m) horizontal. Tanks that require anchorage and tanks conforming to Sec. 14 shall be supported only on Type 1 or 2 foundations (Sec. 12.6.1). Unless otherwise specified, an oiled sand cushion shall be used under the tank bottom. The resistivity of the sand before adding oil should be greater than 3,000 ohm-cm when saturated with distilled or deionized water. Where oiled sand mix is not available or not desired, a compacted cushion of well-graded crushed stone or gravel, clean sand, hydrated-lime–sand mix, asphaltic road mix, or similar material shall be specified. The chloride content of the under-bottom material shall be less than 100 ppm, and the sulfate content shall be less than 200 ppm.

NOTE 1: Oiled sand mixture consists of approximately 18 gal (68 L) of oil per cubic yard (89 L/m^3) of sand. The oils used in the oiled sand mixture shall be either a slow-curing asphalt cutback grade complying with the requirements of ASTM D2026 or a medium-curing asphalt cutback grade complying with the requirements of ASTM D2027. The sand has the correct amount of oil when the sand can be formed into a ball without dripping oil. Sand should be coated but not running with excess oil.

NOTE 2: Hydrated lime may be added to clean sand to obtain a pH of at least 10.5. The resistivity of the sand before hydrated lime is added should be greater than 3,000 ohm-cm when saturated with distilled or deionized water. When the underside of the tank bottom surface is painted, compatibility of the paint with the lime shall be checked with the paint supplier.

12.6.1 *Types of foundations.* Foundations for ground-supported flat-bottom tanks shall be one of the following types:

1. Type 1—Tanks supported on ringwall footings. Tanks may be supported on a ringwall footing under the shell with an oiled sand cushion within the ringwall. The oiled sand cushion inside the ringwall shall be provided above the earthen interior under the tank bottom and shall consist of a minimum of 3-in. (76-mm) cushion of clean sand or fine crushed stone containing an optimum amount of asphalt cutback grade oil as described in Sec. 12.6, NOTE 1. The shell of mechanically anchored tanks shall be supported by grout. For self-anchored tanks where the foundation under the shell meets the tolerances of Sec. 12.6.2, the shell may be supported with ½-in. (13-mm)–thick cane-fiber joint filler meeting the requirements of ASTM D1751. For self-anchored tanks where the foundation

under the shell does not meet the tolerances of Sec. 12.6.2, the shell shall be supported with grout. When grouted, a 1-in. (25-mm) minimum space between the tank bottom and the top of the ringwall shall be grouted full with either a 1:1.5 cement–sand grout or commercial grout, unless otherwise specified. The grout shall extend from the outside edge of the tank bottom to the outside edge of the sand cushion, but in no case shall the width of grout be less than 6 in. (152 mm). The top of the foundation shall be cleaned and thoroughly wetted before grout is placed.

2. Type 2—Tanks supported on concrete slabs. Tanks may be supported on a sand cushion, not less than 1 in. (25 mm) thick between the flat bottom and the concrete slab foundation. In lieu of a sand cushion, the bottom may be supported on ½-in. (13-mm)–thick cane-fiber joint filler meeting the requirements of ASTM D1751. The shell of mechanically anchored tanks shall be supported by grout. For self-anchored tanks where the foundation under the shell meets the tolerances of Sec. 12.6.2, the shell may be supported with ½-in. (13-mm)–thick cane-fiber joint filler meeting the requirements of ASTM D1751. For self-anchored tanks where the foundation under the shell does not meet the tolerances of Sec. 12.6.2, the shell shall be supported with grout. When grouted, a 1-in. (25-mm) minimum space between the tank bottom and the top of the concrete shall be filled with either a 1:1.5 cement–sand grout or commercial grout, unless otherwise specified. The grout shall extend from the outside edge of the tank bottom to the outside edge of the sand cushion or cane-fiber joint filler, but in no case shall the width of grout be less than 6 in. (152 mm). The top of the foundation shall be cleaned and thoroughly wetted before grout is placed.

3. Type 3—Tanks within ringwalls. Tanks may be placed on an oiled sand cushion within a concrete ringwall. The oiled sand cushion shall consist of a minimum of 6-in. (152-mm) cushion of clean sand or fine crushed stone containing an optimum amount of an asphalt cutback grade oil as described in Sec. 12.6, Note 1. The top of the sand within the ringwall should slope uniformly upward from the top of the ringwall to the center of the tank. The inside of the ringwall shall be a minimum of ¾ in. (19 mm) outside the bottom plates of the tank. Adequate provisions for drainage inside the ringwall shall be made.

4. Type 4—Tanks supported on granular berms. Tanks may be supported on a berm without the use of a retainer ring. The berm shall consist of a minimum of 6-in. (152-mm) well-graded crushed stone or gravel. The berm shall extend a minimum of 3 ft (1 m) beyond the tank shell and, from there, have a maximum

slope of 1 vertical to 1.5 horizontal. 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 of the berm shall be provided to ensure against foundation washout, and adequate provisions for drainage of the granular berm shall be made.

5. Type 5—Tanks supported on granular berms with steel retainer rings. Tanks may be supported on a berm inside a steel retainer ring. The berm shall consist of 6 in. (152 mm) of well-graded crushed stone or gravel. The berm shall extend to the steel retainer ring. The steel retainer ring shall be a minimum of 12 in. (305 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. (±3.2 mm) in any 10 ft (3 m) of circumference and within ±½ in. (±13 mm) in the total circumference. Adequate provisions for drainage inside the retainer ring shall be made.

12.6.2 *Tolerances on concrete foundations.* Ringwalls and slabs, after grouting or before placing the cane-fiber joint filler, shall be level within ±⅛ in. (±3.2 mm) in any 30-ft (9.1-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).

12.6.3 *Tolerances on anchor installation.* Unless otherwise specified, design of anchors and anchor attachments shall accommodate, and installation of anchors shall comply with, the following tolerances:

1. Location in plan shall be no more than ¼ in. (6 mm) from the specified location.

2. Projection above top of concrete shall be ±¼ in. (±6 mm) from the specified projection.

3. Vertical misalignment shall not exceed ⅛ in. (3 mm) in 12 in. (305 mm).

Sec. 12.7 Detail Design of Foundations

12.7.1 *Height above ground.* The tops of the concrete foundations shall be a minimum of 6 in. (152 mm) above the finished grade, unless otherwise specified.

12.7.2 *Foundation depth.* The minimum depth of concrete foundations shall be determined from Figure 2. The extreme frost penetration depths in Figure 2 shall be the minimum depth of foundation below the ground line. Foundation depth shall be increased in localities where soil or other factors are favorable to deep frost penetration and may be reduced for piers resting on rock. Consult local records for the extreme frost penetration in the circled area of Figure 2. Uplift

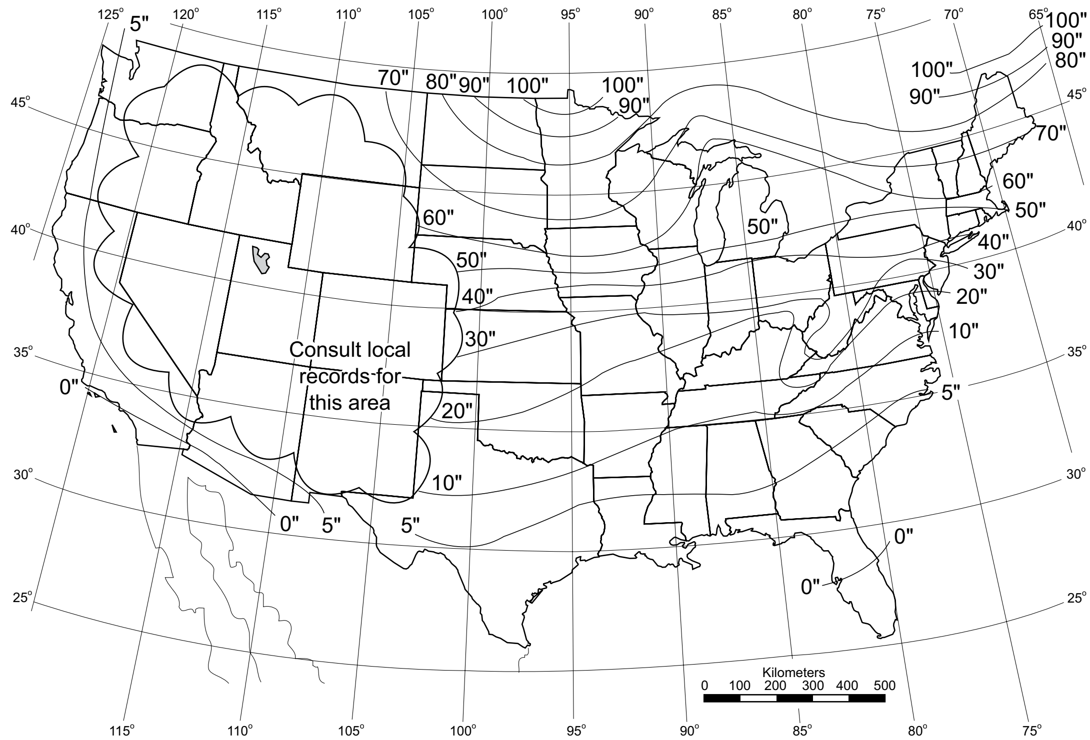


Figure 2 Extreme frost penetration in inches (based on state average)

or soil-bearing requirements may dictate greater depths. Minimum depth of foundation below the ground line shall be 12 in. (300 mm).

12.7.3 *Pile foundations.* If a pile-supported foundation is required, the pile type, pile length below existing grade, and design capacities for gravity loads and gravity loads plus wind or seismic loads shall be specified.

12.7.4 *Buoyancy.* The effect of buoyancy shall be considered when specified.

12.7.5 *Drainage.* The top of the foundation outside the tank or base plate shall be level or sloped to drain away from the tank or base plate.

Sec. 12.8 Concrete Design, Materials, and Construction

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

12.8.1 *Placing concrete.* Unless otherwise specified, riser and column pier concrete shall each be placed monolithically, without any interruption of sufficient duration to permit partial setting of the concrete. If pier concrete is not placed monolithically, a sufficient number of dowels shall be used to transmit all specified loads across the cold joint.

12.8.2 *Finish.* The top portions of piers to a level 6 in. (152 mm) below the proposed ground level shall be finished to a smooth form finish in compliance with ACI 301. Any small holes may be troweled with mortar as soon as possible after the forms are removed.

Sec. 12.9 Backfill

For standpipes and reservoirs with ringwall foundations, all topsoil, organic material, and undesirable material within the ringwall shall be removed and replaced with a controlled, load-bearing backfill. The natural soils and load-bearing backfill within the ringwall shall be capable of supporting the tank bottom without general settlement or localized settlement causing breakdown of the tank bottom adjacent to the ringwall.

12.9.1 *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.

12.9.2 *Pipe cover.* Pipe cover shall be provided in accordance with Figure 3, unless local conditions dictate that greater or lesser cover should be used.

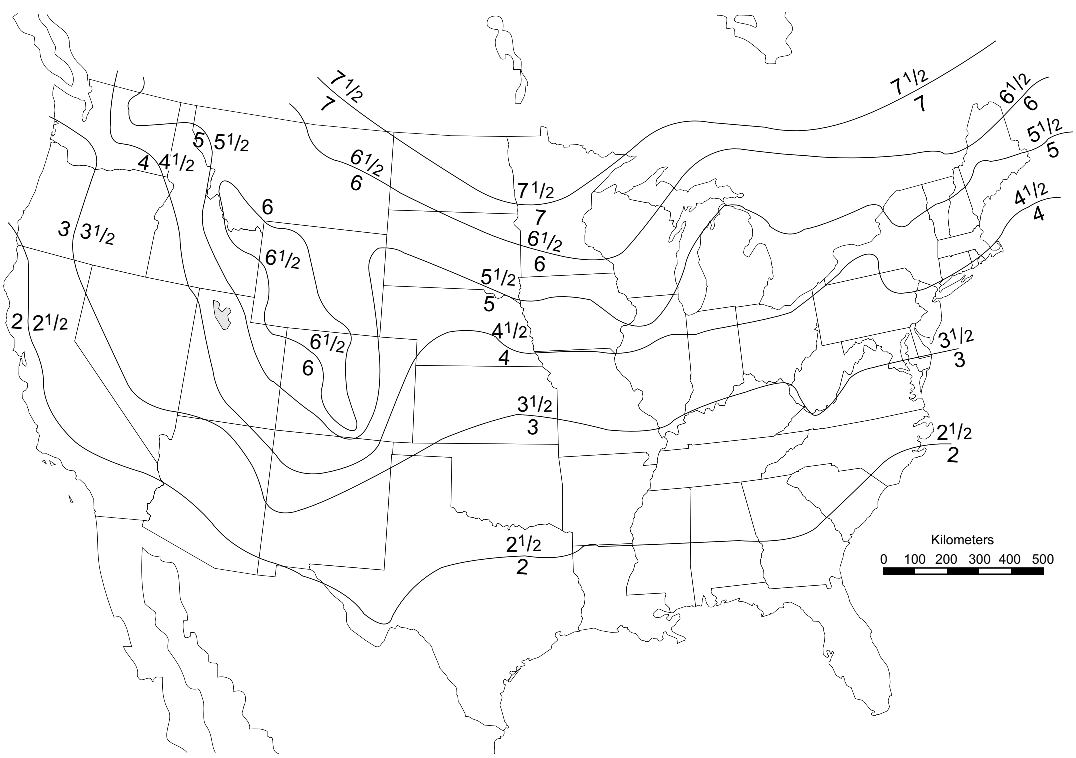


Figure 3 Recommended depth of cover (in feet above top of pipe)

SECTION 13: SEISMIC DESIGN OF WATER STORAGE TANKS

Sec. 13.1 General

13.1.1 *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 caused by an event with a 2 percent probability of 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. 13.2.2) is less than or equal to 0.04 and S_S (Sec. 13.2.2) 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. 13.2.6) or, when specified or required by this standard, the site-specific procedure (Sec. 13.2.7). Seismic design forces are reduced to service-level forces by the use of strength-level to service-level factor, λ_E.

Alternative procedures that account for the effects of soil–structure interaction (elevated tanks and mechanically anchored, ground-supported flat-bottom tanks) and fluid–structure interaction (elevated tanks) are permitted in Sec. 13.2.9 and Sec. 13.2.10.

13.1.2 *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. MCE_R. Risk-adjusted maximum considered earthquake. The most severe earthquake ground motion considered in this standard.

6. Mechanically anchored tanks. Tanks that have anchor bolts or anchor straps to anchor the tank to the foundation.

7. Self-anchored tanks. Ground-supported, flat-bottom tanks that rely on the inherent stability of the self-weight of the tank and contents to resist overturning forces.

8. SRSS. Square root of the sum of the squares.

13.1.3 *Type of structure*. The standard provides seismic design requirements for the following types of structures:

13.1.3.1 Cross-braced column-supported elevated tanks. Cross-braced column-supported elevated tanks rely on truss action of the columns, struts, and diagonal bracing to resist seismic shear and overturning moment. This standard provides seismic design requirements for cross-braced column-supported elevated tanks that use tension-only diagonal bracing. Tanks that use tension-compression diagonal bracing are beyond the scope of this standard.

13.1.3.2 Pedestal-type elevated tanks. Pedestal-type elevated tanks rely on cantilever action of the pedestal to resist seismic shear and overturning moment. Pedestal-type elevated tanks include smooth and bent-plate pedestals.

13.1.3.3 Ground-supported flat-bottom tanks. Ground-supported flat-bottom tanks include reservoirs and standpipes. Ground-supported flat-bottom tanks may be self-anchored or mechanically anchored.

Sec. 13.2 Design Earthquake Ground Motion

13.2.1 *Seismic importance factor, I_E*. The seismic importance factor, I_E, is based on the risk category (Sec. 3.1.1) and shall be in accordance with Table 21.

13.2.2 *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.

Table 21 Seismic importance factor, I_E

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

13.2.3 *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 that Site Class E or F soils are not present at the site.

A site response analysis that complies with Sec. 13.2.7 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. 13.2.4.

13.2.4 *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 22 and 23, respectively.

Table 22 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. 13.2.7) are required.

Table 23 Long-period site coefficient, F_v

Site Class	**Mapped Spectral Response Acceleration at 5 Percent Damping and 1-s Period***				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	†	†	†	†	†

* Use straight-line interpolation for intermediate values of S_1.
† Site-specific evaluation and procedure (Sec. 13.2.7) are required.

Table 24 Response modification factors, R_i and R_c

Structure	Response Modification Factor	
	R_i (Impulsive Component)	R_c (Convective Component)
Cross-braced column-supported elevated tank	3.0*	†
Pedestal-type elevated tank	3.0	†
Ground-supported flat-bottom tank		
Mechanically anchored	3.0	1.5
Self-anchored	2.5	1.5

* The response modification factor, R_i, for cross-braced column-supported elevated tanks only applies to tanks with tension-only diagonal bracing. Tanks that utilize tension-compression diagonal bracing are beyond the scope of this standard.

† For elevated tanks, the design shall be based on components being impulsive unless the alternative fluid–structure interaction procedure is used (Sec. 13.2.9).

13.2.5 *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 impulsive component, R_i, and the response modification factor applied to the convective component, R_c, shall be in accordance with Table 24.

13.2.6 *Design response spectra – general procedure.*

13.2.6.1 General. The general procedure is based on the mapped MCE_R spectral response accelerations from Sec. 13.2.2 for an event with a 2 percent probability of exceedance within a 50-year period.

13.2.6.2 MCE_R spectral response acceleration. Mapped MCE_R spectral response accelerations from Sec. 13.2.2 shall be adjusted for site class effects using the following equations:

$$S_{MS} = F_a S_s \qquad (13\text{-}1)$$

$$S_{M1} = F_v S_1 \qquad (13\text{-}2)$$

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. 13.2.2, 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. 13.2.2, stated as a multiple (decimal) of *g*

F_a = short-period site coefficient from Table 22

F_v = long-period site coefficient from Table 23

g = acceleration due to gravity

13.2.6.3 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}.

$$S_{DS} = US_{MS} \tag{13-3}$$

$$S_{D1} = US_{M1} \tag{13-4}$$

Where:

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

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.

13.2.6.3.1 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 $0 \le T_i \le T_S$: $$S_{ai} = S_{DS} \tag{13-5}$$

For $T_S < T_i \le T_L$: $$S_{ai} = \frac{S_{D1}}{T_i} \le S_{DS} \tag{13-6}$$

For $T_i > T_L$: $$S_{ai} = \frac{T_L S_{D1}}{T_i^2} \tag{13-7}$$

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 22-14 through 22-17

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

The other symbols have been previously defined in this section.

13.2.6.3.2 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 $T_c \le T_L$:
$$S_{ac} = \frac{KS_{D1}}{T_c} \le S_{DS} \tag{13-8}$$

For $T_c > T_L$:
$$S_{ac} = \frac{KT_L S_{D1}}{T_c^2} \tag{13-9}$$

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.

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

13.2.7.1 General. When a site-specific procedure is specified or required by this standard to determine the MCE_R spectral response acceleration, 5 percent damped, at any period S_{aM}, the procedure shall comply with ASCE 7, Chapter 21. The site-specific analysis shall be documented in a report. Refer to Sec. 13.2.3 for requirements for sites classified as Site Class F.

13.2.7.2 Design response spectrum.

13.2.7.2.1 Design response spectrum for impulsive components. The design response spectrum for impulsive components, S_{ai}, shall be based on 5 percent damping and Eq 13-10, except as noted. The design spectral response acceleration determined by Eq 13-10 shall not be less than 80 percent of the design spectral response acceleration by the general procedure (Sec. 13.2.6). 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. 13.2.6).

$$S_{ai} = US_{aM} \tag{13-10}$$

Where:

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

The other symbols have been previously defined in this section.

For ground-supported flat-bottom 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 determined by Eq 13-10 may be limited to (W_T/W_i) (tan 30°) when the tanks are:

1. Self-anchored.
2. Mechanically anchored with anchor bolts and chairs at least 18 in. high and are not otherwise prevented from sliding laterally at least 1 in.

See Sec. 13.5.2.1 and Sec. 13.5.2.2 for definitions of W_T and W_i, respectively.

13.2.7.2.2 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 13-11. The design spectral response acceleration determined by Eq 13-11 shall not be less than 80 percent of the design spectral response acceleration by the general procedure (Sec. 13.2.6).

$$S_{ac} = UKS_{aM} \tag{13-11}$$

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. 13.2.7, except that the damping scaling factor, K, shall be set equal to 1.0.

13.2.8 *Horizontal design accelerations.*

13.2.8.1 Elevated tanks. The design acceleration, A_i, shall be based on the design spectral response acceleration for impulsive components, 5 percent damped, S_{ai} at the natural period of the structure, T_i. The design spectral response acceleration shall be taken from a design spectrum determined by the general procedure (Sec. 13.2.6) or, when specified or required, the site-specific procedure (Sec. 13.2.7). The natural period of the structure shall be determined in accordance with Sec. 13.3.1 for cross-braced column-supported tanks or Sec. 13.4.1 for pedestal-type elevated tanks. The natural period of the structure used to determine the design acceleration shall not exceed 4 s. The design acceleration shall be applied to the total weight of the tank and contents, unless the alternative procedure of Sec. 13.2.9 is used. The design acceleration shall be determined by the equation:

$$A_i = \frac{\lambda_E S_{ai} I_E}{R_i} \geq \frac{0.36 S_1 I_E}{R_i} \qquad (13\text{-}12)$$

Where:

A_i = impulsive design acceleration, stated as a multiple (decimal) of g

I_E = seismic importance factor from Table 21

R_i = response modification factor for the impulsive component from Table 24 for the type of structure

λ_E = strength-level to service-level factor for seismic load

= 0.70

The other symbols have been previously defined in this section.

13.2.8.2 Ground-supported flat-bottom tanks. For the general procedure, the impulsive design acceleration, *A*i, is independent of T_i, and therefore, 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. 13.5.1. The design spectral response accelerations for impulsive and convective components shall be taken from design spectra determined by the general procedure (Sec. 13.2.6) or, when specified or required, the site-specific procedure (Sec. 13.2.7). The impulsive and convective design accelerations shall be determined by the following equations:

$$A_i = \frac{\lambda_E S_{ai} I_E}{R_i} \geq \frac{0.36 S_1 I_E}{R_i} \tag{13-13}$$

$$A_c = \frac{\lambda_E S_{ac} I_E}{R_c} \tag{13-14}$$

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 24 for the type of structure

λ_E = strength-level to service-level factor for seismic load

= 0.70

The other symbols have been previously defined in this section.

13.2.9 *Alternative procedures for elevated tanks.* The effects of fluid–structure and soil–structure interaction may be considered for elevated tanks in accordance with ASCE 7 and the following limitations:

1. 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.

2. Fluid–structure interaction shall not be used when the first mode sloshing wave period is equal to or less than three times the natural period of the structure for the impulsive components.

3. When fluid–structure interaction is used, the sloshing mechanism (i.e., the definition of the convective mass and centroid) shall be determined for the specific tank configuration by detailed fluid–structure interaction analysis or testing. The response modification factor for the convective component, R_c, shall be 1.5, and the convective design acceleration, A_c, shall be determined using Eq 13-14.

4. Base shear including the effects of fluid–structure interaction or soil–structure interaction, or both, V_{alt}, shall not be less than 70 percent of the base shear without the effects of fluid–structure interaction and soil–structure interaction, V. When V_{alt} is less than $0.70V$, shears and overturning moments shall be scaled by the ratio $0.70V/V_{alt}$.

13.2.10 *Alternative procedures for ground-supported flat-bottom tanks.* The effects of soil–structure interaction may be considered for ground-supported flat-bottom tanks 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. Base shear including the effects of soil–structure interaction and fluid–structure interaction, V_{alt}, shall not be less than 70 percent of the base shear including the effects of fluid–structure interaction only, V. When V_{alt} is less than $0.70V$, shears and overturning moments shall be scaled by the ratio $0.70V/V_{alt}$.

Sec. 13.3 Cross-Braced, Column-Supported Elevated Tanks

13.3.1 *Structural period.* The natural period of the structure, T_i, shall be established using the structural properties and deformational characteristics of the resisting elements. The period calculation shall be based on a fixed-base structure with the mass of the structure and contents represented by a single impulsive mass (i.e., the mass moves in unison with the structure). Cross-braced towers can conservatively be assumed to deflect as a guided cantilever (i.e., no rotation at the top). Alternatively, the cross-braced tower may be assumed to deflect as a free cantilever with X_K corrections used to adjust the free cantilever deflection, as shown in the following equation:

$$X_{GC} = X_K X_C \tag{13-15}$$

Where:

X_{GC} = guided cantilever deflection, in consistent units

X_K = correction factor based on the number of columns N:

N	X_K	N	X_K	N	X_K
4	0.379	8	0.436	14	0.422
5	0.429	9	0.421	16	0.421
6	0.451	10	0.429	18	0.420
7	0.421	12	0.425	∞	0.416

X_C = cantilever deflection, in consistent units

13.3.2 *Lateral force.* A lateral force is assumed to act through the center of gravity of the total weight of structure and contents, nonconcurrently, in the direction of each of the main axes of the structure. The lateral force shall be determined by the equation:

$$V = A_i W \qquad (13\text{-}16)^*$$

Where:

V = lateral force, in pounds

A_i = design acceleration from Eq 13-12, stated as a multiple (decimal) of g

W = total weight of structure and contents, in pounds

13.3.3 *Seismic design requirements.*

13.3.3.1 Tower stresses. Calculated stress levels in the tower shall not exceed the allowable design stresses set forth in ANSI/AISC 360, increased by one-third for seismic loading.

13.3.3.2 P-delta effects. The design of the tower, anchorage, and foundation shall include P-delta effects from a lateral drift equal to three times the elastic deflection, measured at the centroid of the contents, caused by the horizontal design acceleration, A_i.

13.3.3.3 Ductility. A separate calculation shall be made to show that a stress equal to 4⁄3 times the minimum published yield stress can be developed in the bracing without failure of the connections, wing plates, struts, or anchor bolts.

13.3.3.4 Foundation stability. Foundations shall be checked for stability using a lateral seismic force sufficient to yield the bracing rods. Ultimate bearing capacity shall be permitted in this case (Figure 4).

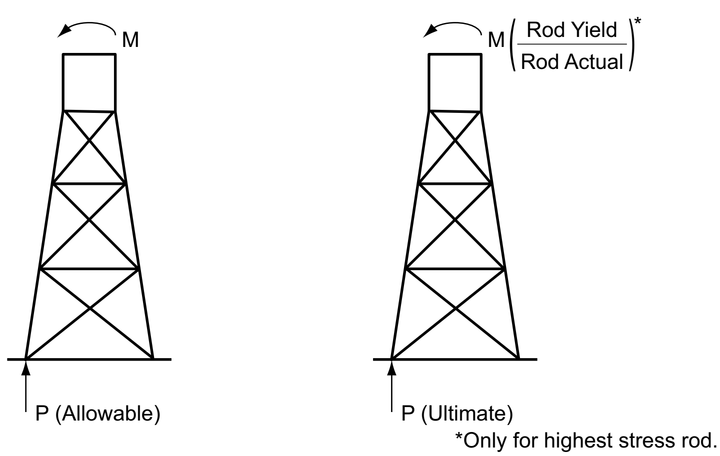


Figure 4 Diagram for checking foundation stability of cross-braced, column-supported elevated tanks

* For equivalent metric equation, see Sec. 13.9.

13.3.3.5 Upset bracing rods. All threaded seismic shear-resisting bracing rods shall have upset or enlarged thread ends, with root area greater than the bracing rod area.

13.3.3.6 Vertical design acceleration. The design of the tank, tower, and anchorage shall include load effects from vertical design acceleration, A_v, equal to $0.48S_{DS}$ for responses limited by buckling and $0.19S_{DS}$ for responses not limited by buckling. 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 the vertical design acceleration being multiplied by 0.40, or by the SRSS method, unless otherwise specified.

13.3.3.7 Freeboard. Freeboard shall be in accordance with Sec. 13.6.

Sec. 13.4 Pedestal-Type Elevated Tanks

13.4.1 *Structural period*. The natural period of the structure, T_i, shall be established using the structural properties and deformational characteristics of the resisting elements. The period calculation shall be based on a fixed-base structure with the mass of the structure and contents represented by a single impulsive mass (i.e., the mass moves in unison with the structure).

13.4.2 *Lateral force*. A lateral force is assumed to act through the center of gravity of the total weight of the structure and contents. The lateral force shall be determined by the equation:

$$V = A_i W \qquad (13\text{-}17)^*$$

Where:

V = lateral force, in pounds

A_i = design accelerations from Eq 13-12, stated as a multiple (decimal) of g

W = total weight of structure and contents, in pounds

13.4.3 *Seismic design requirements*.

13.4.3.1 Pedestal stresses. Compressive stresses shall not exceed the buckling stress allowed by Sec. 3.4, increased by one-third for seismic loading. Tensile stresses shall be limited to basic allowable tensile stress times joint efficiency, increased by one-third for seismic loading.

* For equivalent metric equation, see Sec. 13.9.

13.4.3.2 P-delta effects. The design of the pedestal, anchorage, and foundation shall include P-delta effects from a lateral drift equal to three times the elastic deflection, measured at the centroid of the contents, caused by the horizontal design acceleration, A_i.

13.4.3.3 Vertical design acceleration. The design of the tank, pedestal, and anchorage shall include load effects from vertical design acceleration, A_v, equal to $0.48S_{DS}$ for responses limited by buckling and $0.19S_{DS}$ for responses not limited by buckling. 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.

13.4.3.4 Freeboard. Freeboard shall be in accordance with Sec. 13.6.

13.4.3.5 Critical buckling check. Shell elements that resist horizontal seismic forces and whose primary mode of failure is local or general buckling shall be checked to ensure that buckling will not occur when load effects from horizontal design acceleration with $\lambda_E I_E/\mathrm{R}_i$ equal to 1.0 (i.e., $A_i = S_{ai}$) are considered. The structural response may include the effects of soil–structure and fluid–structure interaction. The base shear and overturning moment, including the effects of soil–structure and fluid–structure interaction, shall not be less than 50 percent of the corresponding values without the effects of soil–structure and fluid–structure interaction. The buckling resistance shall be taken as the critical buckling capacity of the shell element (i.e., factor of safety equal to 1.0). The critical check shall include effects from lateral drift, measured at the centroid of the contents, equal to the elastic displacement caused by the horizontal design acceleration with $\lambda_E I_E/R_i$ equal to 1.0. Vertical design acceleration need not be considered when making the critical buckling check. The critical buckling capacity of shell elements may be determined by increasing the values of F_L obtained in Sec. 3.4.3 by a factor of 2 ($2F_L \leq F_y$), with appropriate slenderness reduction factors applied.

Sec. 13.5 Ground-Supported Flat-Bottom Tanks

13.5.1 *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.

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:

$$T_c = 2\pi\sqrt{\frac{D}{3.68g\tanh\left(\frac{3.68H}{D}\right)}} \tag{13-18}$$

Where:

T_c = first mode sloshing wave period, in seconds

D = tank diameter, in feet

H = distance from bottom of shell to MOL, in feet

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

13.5.2.1 Horizontal design acceleration. 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

$$M_s = \sqrt{\left[A_i\left(W_sX_s + W_rH_t + W_iX_i\right)\right]^2 + \left[A_cW_cX_c\right]^2} \quad \text{(13-19)*}$$

Where:

M_s = design overturning moment at the bottom of the shell caused by horizontal design acceleration, in foot-pounds

A_i = impulsive design acceleration from Eq 13-13, stated as a multiple (decimal) of g

A_c = convective design acceleration from Eq 13-14, 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 roof knuckle, plus permanent loads, if specified, in pounds

* For equivalent metric equation, see Sec. 13.9.

W_i = weight of effective mass of tank contents that moves in unison with the tank shell (effective impulsive weight), in pounds (Sec. 13.5.2.2)

W_c = weight of effective mass of the first mode sloshing contents of the tank (effective convective weight), in pounds (Sec. 13.5.2.2)

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. 13.5.2.2)

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. 13.5.2.2)

13.5.2.2 Effective weight of tank contents.

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

For $D/H \geq 1.333$:

$$W_i = \frac{\tanh\left(0.866\frac{D}{H}\right)}{0.866\frac{D}{H}} W_T \tag{13-20}$$

For $D/H < 1.333$:

$$W_i = \left[1.0 - 0.218\frac{D}{H}\right] W_T \tag{13-21}$$

For all proportions of D/H:

$$W_c = 0.230\frac{D}{H}\tanh\left(\frac{3.67H}{D}\right) W_T \tag{13-22}$$

Where:

W_T = total weight of tank contents, in pounds, determined by the equation:

$$W_T = 62.4GH\left(\frac{\pi D^2}{4}\right) = 49GHD^2 \tag{13-23}*$$

Where:

G = specific gravity = 1.0 for water

The other symbols have been previously defined in this section.

13.5.2.2.2 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 $D/H \geq 1.333$:

$$X_i = 0.375H \tag{13-24}$$

For $D/H < 1.333$:

$$X_i = \left[0.5 - 0.094\frac{D}{H}\right]H \tag{13-25}$$

For all proportions of D/H:

$$X_i = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \tag{13-26}$$

The symbols have been previously defined in this section.

13.5.2.2.3 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.

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

13.5.3.1 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:

$$V_f = \sqrt{\left[A_i\left(W_s + W_r + W_f + W_i\right)\right]^2 + \left[A_cW_c\right]^2} \tag{13-27}*$$

* For equivalent metric equation, see Sec. 13.9.

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.

13.5.3.2 Design overturning moment at the top of the foundation.

13.5.3.2.1 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 13-19.

13.5.3.2.2 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:

$$M_{mf} = \sqrt{\left[A_i\left(W_s X_s + W_r H_t + W_i X_{imf}\right)\right]^2 + \left[A_c W_c X_{cmf}\right]^2} \quad (13\text{-}28)^*$$

For $D/H \geq 1.333$:

$$X_{imf} = 0.375\left[1.0 + 1.333\left(\frac{0.866\frac{D}{H}}{\tanh\left(0866\frac{D}{H}\right)} - 1.0\right)\right]H \quad (13\text{-}29)$$

For $D/H < 1.333$:

$$X_{imf} = \left[0.50 + 0.06\frac{D}{H}\right]H \quad (13\text{-}30)$$

For all proportions of D/H:

$$X_{cmf} = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1.937}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \quad (13\text{-}31)$$

* For equivalent metric equation, see Sec. 13.9.

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 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 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.

13.5.4 *Seismic design requirements.*

13.5.4.1 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 weight of a portion of the tank contents adjacent to the shell for self-anchored tanks, or by mechanically anchoring the tank shell.

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

1. The overturning ratio, J, determined by Eq 13-32 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. 13.5.4.2.
3. The required thickness of the bottom annulus, t_b, does not exceed the thickness of the bottom shell ring per Sec. 13.5.4.1.2.
4. Piping flexibility requirements of Sec. 13.7 are satisfied.

The overturning ratio, J, is given by the equation:

$$J = \frac{M_s}{D^2\left[w_t\left(1 - 0.4A_v\right) + w_L\right]} \qquad \text{(13-32)*}$$

Where:

J = overturning ratio

w_t = weight of the tank shell and portion of the roof reacting on the shell, determined by Eq 13-37, in pounds per foot of shell circumference

* For equivalent metric equation, see Sec. 13.9.

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. 13.5.4.1.1)

A_v = vertical design acceleration (Sec. 13.5.4.3), 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 because of 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 that the shell compression requirements of Sec. 13.5.4.2 are satisfied.

For $J > 1.54$, the tank is not stable. Modify the bottom annulus, within the limits of t_b (Sec. 13.5.4.1.1) and L (Sec. 13.5.4.1.2) or provide mechanical anchors.

13.5.4.1.1 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 a separate butt-welded annular plate. For self-anchored tanks, the resisting force of the bottom annulus shall be determined by the equation:

$$w_L = 7.9 t_b \sqrt{F_y HG} \leq 1.28HDG \qquad (13\text{-}33)^*$$

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

The other symbols have been previously defined in this section.

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

13.5.4.1.2 The bottom annulus may be thicker than the bottom shell course, but the thickness, t_b, used to calculate seismic 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:

$$L = 0.216 t_b \sqrt{\frac{F_y}{HG}} \text{ in feet } \leq 0.035D \qquad (13\text{-}34)^*$$

* For equivalent metric equation, see Sec. 13.9.

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 mechanically anchored. When a butt-welded annulus is used, the width of the butt-welded annulus measured from the inside of the shell shall not be less than 18 in. (457 mm).

13.5.4.2 Shell stresses.

13.5.4.2.1 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:

$$\sigma_c = \left[w_t \left(1 + 0.4A_v\right) + \frac{1.273M_s}{D^2} \right] \frac{1}{12t_s} \quad (13\text{-}35)^*$$

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

$$\sigma_c = \left[\frac{w_t \left(1 + 0.4A_v\right) + w_L}{0.607 - 0.18667J^{2.3}} - w_L \right] \frac{1}{12t_s} \quad (13\text{-}36)^*$$

In Eqs 13-35 and 13-36,

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

$$w_t = \frac{W_s}{\pi D} + w_{rs} \quad (13\text{-}37)^*$$

* For equivalent metric equation, see Sec. 13.9.

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 or equal to the seismic allowable stress, σ_e, which is determined in accordance with Sec. 13.5.4.2.4.

13.5.4.2.2 Longitudinal shell compression for mechanically anchored tanks. When mechanical anchors are provided, the maximum longitudinal compression stress at the bottom of the shell shall be determined by Eq 13-35.*

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

$$\sigma_s = \frac{\sqrt{N_i^2 + N_c^2 + \left(N_b A_v\right)^2}}{t_s} \qquad (13\text{-}38a)^{\dagger}$$

Or

$$\sigma_s = \frac{\sqrt{N_i^2 + N_c^2} + 0.4\left(N_b A_v\right)}{t_s} \qquad (13\text{-}38b)^{\dagger}$$

For $D/H \geq 1.333$:

$$N_i = 4.5 A_i G D H \left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right] \tanh\left(0.866\frac{D}{H}\right) \qquad (13\text{-}39)^{\dagger}$$

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

$$N_i = 2.77 A_i G D^2 \left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right] \qquad (13\text{-}40)^{\dagger}$$

* The application of Eq 13-35 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.

† For equivalent metric equations, see Sec. 13.9.

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

$$N_i = 1.39 A_i G D^2 \tag{13-41*}$$

For all proportions of D/H:

$$N_c = \frac{0.98 A_c G D^2 \cosh\left[\frac{3.68(H-Y)}{D}\right]}{\cosh\left[\frac{3.68H}{D}\right]} \tag{13-42*}$$

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, in pounds per inch = $2.6GYD$
- 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 (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.

13.5.4.2.4 Allowable shell stress. Allowable shell plate stresses in tension for the material of construction shall be based on the allowable stress in Sec. 3 or Sec. 14, as applicable. The allowable stress shall be reduced by the applicable joint efficiency (Table 11 or Sec. 14, if applicable). A one-third increase in basic allowable stress is permitted for seismic loading.

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:

$$\sigma_e = 1.333\left(\sigma_a + \frac{\Delta\sigma_{cr}}{2}\right) \tag{13-43}$$

* For equivalent metric equations, see Sec. 13.9.

For mechanically anchored tanks:

$$\sigma_e = 1.333\sigma_a \tag{13-44}$$

Where:

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

σ_a = allowable compression stress, F_L, from Sec. 3.4.3.1, in pounds per square inch

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

$$\Delta\sigma_{cr} = \frac{\Delta C_c E t}{R} \qquad \text{(13-45)*}$$

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

$$\text{For } \frac{P}{E}\left(\frac{R}{t}\right)^2 \le 0.064 \qquad \Delta C_c = 0.72\left[\frac{P}{E}\left(\frac{R}{t}\right)^2\right]^{0.84} \qquad \text{(13-46)*}$$

$$\text{For } \frac{P}{E}\left(\frac{R}{t}\right)^2 \le 0.064 \qquad \Delta C_c = 0.045\ln\left[\frac{P}{E}\left(\frac{R}{t}\right)^2 + 0.0018\right] + 0.194 \le 0.22 \qquad \text{(13-47)*}$$

Where:

ln = natural logarithm function.

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

13.5.4.3 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 (Eqs 13-35 and 13-36) and $0.19S_{DS}$ for responses

* For equivalent metric equation, see Sec. 13.9.

not limited by buckling (Eqs 13-32, 13-38a, and 13-38b), except as permitted in Sec. 13.5.4.5. 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.

13.5.4.4 Freeboard. Freeboard shall be in accordance with Sec. 13.6.

13.5.4.5 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 not be used to reduce uplift caused by overturning (Eq 13-37). 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 on the allowable stress design provisions of ANSI/AISC 360, increased one-third for seismic loading.

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

$$V_{ALLOW} = \lambda_E \tan 30^\circ \left[W_s + W_r + W_i + W_c\right]\left(1 - 0.4A_v\right) \tag{13-48}$$ *

Where:

V_{ALLOW} = allowable lateral shear, in pounds

λ_E = strength-level to service-level factor for seismic load

= 0.70

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 13-27, or additional shear resistance with a capacity of at least V_{NET} (see Sec. 3.8.6.1) must be provided.

Sec. 13.6 Freeboard

Sloshing of the tank contents 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 Eq 13-49.

$$d = 0.42DA_f \tag{13-49}$$

* For equivalent metric equation, see Sec. 13.9.

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 the following equations:

For Risk Categories II and III:

When $T_c \leq 4$:

$$A_f = \frac{KS_{D1}I_E}{T_c} \tag{13-50}$$

When $T_c > 4$:

$$A_f = \frac{4KS_{D1}I_E}{T_c^2} \tag{13-51}$$

For Risk Category IV:

When $T_c \leq T_L$:

$$A_f = \frac{KS_{D1}}{T_c} \tag{13-52}$$

When $T_c > T_L$:

$$A_f = \frac{KS_{D1}T_L}{T_c^2} \tag{13-53}$$

Where:

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 21

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 22-14 through 22-17

The other symbols have been previously defined in this section.

The design freeboard shall be determined by the requirements of Table 25. If the freeboard provided is less than the design freeboard, the water masses shall be adjusted to include the confined portion of the sloshing mass as additional

Table 25 Design freeboard requirements

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

* No minimum freeboard is required.

impulsive mass. The roof and supporting structure shall be designed to contain the sloshing liquid and to resist the pressure of an equivalent hydrostatic head equal to the design freeboard less the freeboard provided.

Sec. 13.7 Piping Connections

13.7.1 *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 designed for the seismic displacements and defined operating pressure.

Unless otherwise specified, piping systems shall provide for the minimum design displacements in Table 26 at working stress levels (with one-third allowable stress increase for seismic loads) in the piping, supports, and tank connection. The values given in Table 26 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 26 may be reduced to 70 percent of the values shown.

13.7.2 *Bottom connection for self-anchored, ground-supported flat-bottom tanks.* The bottom connection for a self-anchored, ground-supported flat-bottom 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 13-34 plus 12 in. (305 mm), as shown in Figure 5.

Table 26 Minimum design displacements for piping attachments

Condition	Displacement in.	Displacement (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

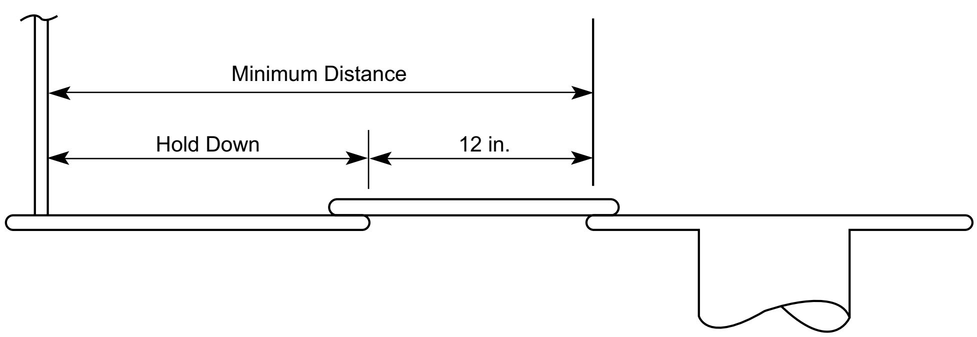


Figure 5 Bottom piping connection of a self-anchored, ground-supported flat-bottom tank

Sec. 13.8 Foundation Design for Ground-Supported Flat-Bottom Tanks

13.8.1 *Mechanically anchored, ground-supported flat-bottom tanks.* Ringwalls and footings for mechanically anchored tanks shall be proportioned to resist maximum anchor bolt uplift and overturning bearing pressure. Water load directly over the ringwall and footing may be used to resist the maximum anchor bolt uplift, provided that the ringwall and footing are designed to carry this eccentric loading. Water load shall not be used to reduce the anchor bolt load.

13.8.2 *Self-anchored, ground-supported flat-bottom tanks.* Design shell compression loads on the foundation should be determined using the same method as for the mechanically anchored tank condition (Eq 13-35). This assumption does not permit larger response modification factors R_i and R_c than permitted for self-anchored tanks.

13.8.3 *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. 13.5.4.2.1 or Sec 13.5.4.2.2 as appropriate.

Sec. 13.9 Equivalent Metric Equations

Metric equivalents of equations presented in Sec. 13 are as follows:

Equation Number	Equivalent Metric Equation	Variable	Metric Units
13-16 13-17	$V = 9.81A_iW$	V W	N kg
13-19	$M_s = 9.81\sqrt{\left[A_i\left(W_sX_s + W_rH_t + W_iX_i\right)\right] + \left[A_cW_cX_c\right]}$	M_s W_s, W_r W_i, W_c X_s, H_t X_i, X_c	N-m kg kg m m
13-23	$W_T = 785.4GHD^2$	W_T D, H	kg m
13-27	$V_f = 9.81\sqrt{\left[A_i\left(W_s + W_r + W_f + W_i\right)\right]^2 + \left[A_cW_c\right]^2}$	V_f W_s, W_r W_f, W_i W_c	N kg kg kg
13-28	$M_{mf} = 9.81\sqrt{\left[A_i\left(W_sX_s + W_rH_t + W_iX_{imf}\right)\right]^2 + \left[A_cW_cX_{cmf}\right]^2}$	M_{mf} W_s, W_r W_i, W_c X_s, H_t X_{imf}, X_{cmf}	N-m kg kg m m
13-32	$J = \dfrac{M_s}{D^2\left[w_t\left(1 - 0.4A_v\right) + w_L\right]}$	M_s D w_t, w_L	N-m m N/m
13-33	$w_L = 99t_b\sqrt{F_yHG} \le 201.1HDG$	w_L t_b F_y H, D	N/m mm MPa m
13-34	$L = 0.0172t_b\sqrt{\dfrac{F_y}{HG}} \le 0.035D$	L, D t_b F_y	m mm MPa

13-35	$\sigma_c = \left[w_t\left(1 + 0.4A_v\right) + \frac{1.273M_s}{D^2}\right]\left[\frac{1}{1{,}000t_s}\right]$	σ_c w_t M_s D t_s	MPa N/m N-m m mm
13-36	$\sigma_c = \left[\frac{w_t\left(1 + 0.4A_v\right) + w_L}{0.607 - 0.18667J^{2.3}} - w_L\right]\left[\frac{1}{1{,}000t_s}\right]$	σ_c w_t, w_L t_s	MPa N/m mm
13-37	$w_t = \frac{9.81W_s}{\pi D} + w_{rs}$	w_t, w_{rs} W_s D	N/m kg m
13-38a	$\sigma_s = \frac{\sqrt{N_i^2 + N_c^2 + \left(N_b A_v\right)^2}}{1{,}000t_s}$	σ_s N_i, N_c, N_b t_s	MPa N/m mm
13-38b	$\sigma_s = \frac{\sqrt{N_i^2 + N_c^2} + 0.4\left(N_b A_v\right)}{1{,}000t_s}$	σ_s N_i, N_c, N_b t_s	MPa N/m mm
13-39	$N_i = 8{,}480A_iGDH\left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right]\tanh\left(0.866\frac{D}{H}\right)$	N_i D, H, Y	N/m m
13-40	$N_i = 5{,}220A_iGD^2\left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right]$	N_i D, Y	N/m m
13-41	$N_i = 2{,}620A_iGD^2$	N_i D	N/m m
13-42	$N_c = \frac{1{,}850A_cGD^2\cosh\left[\frac{3.68(H - Y)}{D}\right]}{\cosh\left[\frac{3.68H}{D}\right]}$	N_c D, H, Y	N/m m
13-45	$\Delta\sigma_{cr} = \frac{\Delta C_c Et}{R}$	$\Delta\sigma_{cr}$, E t, R	MPa mm
13-46	For $\frac{P}{E}\left(\frac{R}{t}\right)^2 \le 0.064 \qquad \Delta C_c = 0.72\left[\frac{P}{E}\left(\frac{R}{t}\right)^2\right]^{0.84}$	P, E t, R	MPa mm
13-47	For $\frac{P}{E}\left(\frac{\mathrm{R}}{t}\right)^2 \le 0.064$: $\Delta C_c = 0.045\ln\left[\frac{P}{E}\left(\frac{\mathrm{R}}{t}\right)^2 + 0.0018\right] + 0.194 \le 0.22$	P, E t, R	MPa mm
13-48	$V_{ALLOW} = 9.81\lambda_E \tan 30^\circ\left[W_s + W_r + W_i + W_c\right]\left(1 - 0.4A_v\right)$	V_{ALLOW} W_s, W_r W_i, W_c	N kg kg

SECTION 14: ALTERNATIVE DESIGN BASIS FOR STANDPIPES AND RESERVOIRS

Sec. 14.1 Alternative Design Basis

This section provides rules, design stresses, and inspection requirements to ensure suitable designs and workmanship and the proper temperature ranges for economical use of the higher quality steels with improved notch toughness. Mixing Sec. 3 and Sec. 14 maximum design allowable tensile stress for shell plates in contact with water is not permitted by this standard.

14.1.1 *Applicability.* This alternative design basis is applicable only to ground-supported flat-bottom tanks and only when specified. Tanks designed on this basis must incorporate provisions of this section. When this alternative design basis is used, the requirements of Sec. 2.2.3.2 and Sec. 3 are superseded by any differing requirements of Sec. 14. Other requirements of Sec. 1 through Sec. 13 shall apply. Category 3 materials (see Table 29) shall not be used where the mapped spectral response acceleration at 1-s period, S_1, is greater than 0.15g and the mapped spectral response acceleration at 0.2-s period, S_S, is greater than 0.3g.

14.1.2 *Exclusions.* This section shall not be applied in whole or in part to the design or construction of elevated water tanks.

14.1.3 *Inspections.* The tank shall be inspected and tested in accordance with Sec. 14.4.

14.1.4 *Welding procedure qualification.* Welding procedure qualification shall be made in accordance with the requirements of Sec. 8.2.1. In addition, supplementary essential variables for toughness in ASME BPVC Sec. IX or AWS B2.1/B2.1M apply to shell and annular butt joints. Impact testing shall be performed for each welding procedure qualification. Impact testing, as used herein, shall mean Charpy V-notch impact testing and shall be in accordance with Sec. 14.1.5.

14.1.5 *Impact testing.* Impact testing, as used herein, shall mean Charpy V-notch impact testing and shall be in accordance with ASTM A20/A20M and A370 using Charpy V-notch specimens. The test temperature shall be the design metal temperature specified in Sec. 14.2.4 for category 1 and 2 materials. The test temperature shall be the lesser of the design metal temperature specified in Sec. 14.2.4 and 32°F (0°C) for category 3 material. Each impact test shall consist of three specimens taken from a single test coupon or test location. Acceptance criteria will be the minimum average value at the test temperature. Except for quenched and tempered material, impact tests shall show a minimum average impact energy value of 15 ft-lbf (20.3 N-m) at the design metal temperature. Except for category 3 material, impact tests on quenched and tempered material shall show a minimum average impact energy value of 20 ft-lbf (27 N-m). Impact tests on category 3 material shall show an average of at least 15-mils (381-μm) lateral expansion at the test temperature. The impact energy values obtained on sub-size specimens shall not be less than the values stated above, multiplied by the ratio of the thickness of sub-size specimens to the thickness of full-size specimens.

14.1.5.1 Specimen size. Specimens shall be Charpy V-notch (Type A), in accordance with ASTM A370. When sub-size specimens must be used, the sub-size specimens shall have a width along the notch of at least 80 percent of the material thickness.

14.1.5.2 Weld-metal impact testing. Weld-metal specimens shall be taken across the weld with the notch in the weld metal. The specimens shall be oriented so that the notch is perpendicular to the surface of the material. One face of the specimen shall be substantially parallel to and within 1⁄16 in. (1.6 mm) of the surface. The impact test on the weld metal used to join category 1 or 2 materials shall meet the minimum average impact energy value set forth in Sec. 14.1.5 for the type of material being joined. The impact test on the weld metal used to join category 3 material shall meet the minimum average lateral expansion value set forth in Sec. 14.1.5.

14.1.5.3 Base-metal impact testing for category 1 and 2 materials. When base-metal impact testing is required (Sec. 14.2, Sec. 14.2.1, Sec. 14.2.2, and Sec. 14.3.2.3), a transverse impact test shall be performed for each as-rolled plate after heat treatment, if any was done. The specimens shall be oriented so that the notch is perpendicular to the surface of the plate. The impact tests shall meet the minimum average impact energy values for the type of material being joined, as set forth in Sec. 14.1.5 and as modified by Sec. 14.3.2.3.

14.1.5.4 Base-metal impact testing for category 3 material. A transverse impact test shall be performed for each as-rolled plate after heat treatment. The specimens shall be oriented so that the notch is perpendicular to the surface of the plate. The impact tests shall meet the minimum average lateral expansion value set forth in Sec. 14.1.5.

14.1.5.5 Heat-affected zone impact testing. When base-metal impact testing is required for category 1 or 2 materials, an impact test shall be performed on the heat-affected zone of each vertical seam welding procedure qualification test plate. An impact test shall be performed on the heat-affected zone of all welding procedure qualification test plates for category 3 material. The specimens shall be taken across the weld and as near the surface of the material as is practical. The specimens shall be of sufficient length to locate, after etching, the notch in the heat-affected zone. The notch shall be approximately perpendicular to the surface of the material and located to include as much heat-affected zone material as possible in the resulting fracture. The impact test on the heat-affected zone for category 1 or 2 materials shall meet the minimum average impact energy values for the type of material being joined, as set forth in Sec. 14.1.5. The impact test on the heat-affected zone for category 3 material shall meet the minimum average lateral expansion value set forth in Sec. 14.1.5.

Sec. 14.2 Materials

Plate materials for shell plates, insert plates, intermediate stiffeners, penetrations and their reinforcements, and anchor-bolt chairs are to comply with this section and are classified according to the following three categories: category 1, low-strength material; category 2, medium-strength material; and category 3, high-strength ASTM A517/A517M material. These materials shall be limited to the thickness stated for the design metal temperatures shown in Tables 27 through 29, except that materials listed in these three categories may be used at colder design metal temperatures, provided that the impact requirements specified in Sec. 14.1.4 and Sec. 14.1.5 are met. Additionally, category 1 and 2 materials may be used at thicknesses greater than tabulated for the design metal temperature, provided that the impact requirements in Sec. 14.1.4 and Sec. 14.1.5 are met and provided that the shell plate thickness does not exceed 1.5 in. (38.1 mm). Materials produced to specifications other than those listed may be used, provided that they are certified to meet all requirements of a material listed herein. Piping used for shell penetrations and their reinforcement shall comply with the requirements of Sec. 14.2.6.5. The materials used for the top angle of the shell, the roof and roof knuckle, the bottom plates, and other attachments and appurtenances not listed above shall conform to the requirements of Sec. 2.

Table 27 Category 1 material requirements

Maximum Plate Thickness		Permissible Minimum Specifications		Maximum Insert Plate Thickness*	
in.†	*(mm)*	**Specification**	**Grade**	*in.*	*(mm)*
		+20°F (–6.7°C) or warmer			
½	(13)	ASTM A283/A283M	Gr C		
		ASTM A131/A131M	Gr A		
		ASTM A36/A36M			
1	(25)	ASTM A131/A131M	Gr B		
		ASTM A36/A36M, modified‡			
		CAN/CSA G40.21	Gr 44W, 38W		
		–10°F (–23.3°C) or warmer			
½	(13)	ASTM A131/A131M	Gr B		
		ASTM A36/A36M, modified‡			
		CAN/CSA G40.21	Gr 44W, 38W		
1½	(38)	CAN/CSA G40.21	Gr 44T, 38T		
		ASTM A662/A662M	Gr B		
		ASTM A573/A573M	Gr 58		
		ASTM A516/A516M	Gr 60 norm	2½	(64)
		–40°F (–40°C) or warmer			
½	(13)	CAN/CSA G40.21	Gr 44T, 38T		
		ASTM A662/A662M	Gr B		
		ASTM A573/A573M	Gr 58		
		ASTM A516/A516M	Gr 60		
1½	(38)	ASTM A516/A516M	Gr 60 norm	2½§	(64)
		CAN/CSA G40.21	Gr 44T, 38T norm		
		ASTM A662/A662M	Gr B norm	2	(51)
		ASTM A573/A573M	Gr 58 norm		

* Maximum insert plate thickness is the same as the stated maximum shell plate thickness for each grade unless greater thickness is shown in the table.

† Including corrosion allowance. Permissible range without impact testing.

‡ ASTM A36/A36M, modified. Materials shall conform to ASTM A36/A36M with the following special requirements: (1) plates must be semi-killed or fully killed and (2) the manganese content shall be 0.80%–1.20% by heat analysis except that, for thicknesses greater than 0.75 in., the manganese content may be modified as follows: for each reduction of 0.01% below the specified carbon maximum, an increase of 0.06% manganese above the specified maximum will be permitted up to the maximum of 1.35%.

§ These are the only insert plates exempt from impact testing if used at –10°F (–23.3°C) or warmer design metal temperatures.

A nameplate shall be furnished to identify a tank designed and constructed in accordance with this section. The nameplate shall include the following information:

1. AWWA D100-_____, including Sec. 14;

(*Note: The constructor is to fill in the applicable edition year of the standard under which the tank was designed.*)

2. constructor;
3. year completed;
4. contract number;
5. nominal capacity, gal;
6. nominal diameter, ft;
7. MOL, ft;
8. design metal temperature;
9. shell material; and
10. heat treatment.

14.2.1 *Category 1.* Category 1 materials are listed in Table 27. Base-metal impact tests meeting the requirements of Sec. 14.1.5 shall be provided for designs outside the temperature and thickness ranges of Table 27. Base-metal impact tests are not required for designs within the ranges of the table. Certified records that the weld qualification procedures meet the requirements of Sec. 14.1.4 shall be provided.

14.2.2 *Category 2.* Category 2 materials are listed in Table 28. Base-metal impact tests meeting the requirements of Sec. 14.1.5 shall be provided for designs outside the temperature and thickness ranges of Table 28. Base-metal impact tests are not required for designs within the ranges of the table. Certified records that the weld qualification procedures meet the requirements of Sec. 14.1.4 shall be provided.

14.2.3 *Category 3.* Category 3 materials are listed in Table 29. The base-metal, weld-metal, and heat-affected zones shall be impact tested in accordance with Sec. 14.1.5.

Table 28 Category 2 material requirements

Group	Specification	Grade	Maximum Insert Thickness* *in. (mm)*	Lowest Design Metal Temperature for Shell Plates Without Impact Testing *°F (°C)*
1[†]	ASTM A573/A573M	Gr 70	—	+20 (–6.7) for $t \le 1\frac{1}{2}$ in. (38 mm)
	ASTM A588/A588M		—	+5 (–15.0) for $t \le 1$ in. (25 mm)
	ASTM A516/A516M	Gr 70	—	–10 (–23.3) for $t \le \frac{1}{2}$ in. (13 mm)
	ASTM A662/A662M	Gr C	—	
2[‡]	ASTM A573/A573M	Gr 70	—	–10 (–23.3) for $t \le 1\frac{1}{2}$ in. (38 mm)
	ASTM A516/A516M	Gr 70	2½ (64)	–25 (–31.7) for $t \le 1$ in. (25 mm)
				–40 (–40) for $t \le \frac{1}{2}$ in. (13 mm)
	ASTM A633/A633M	Gr C, D	2½ (64)	–20 (–28.9) for $t \le 1\frac{1}{2}$ in. (38 mm)
	ASTM A537/A537M	Cl 1	2½ (64)	–30 (–34.4) for $t \le 1$ in. (25 mm)
	ASTM A662/A662M	Gr C	—	–40 (–40) for $t \le \frac{1}{2}$ in. (13 mm)
3[§]	ASTM A537/A537M	Cl 2	2½ (64)	

* When no thickness is listed under the "Maximum Insert Thickness" header, the maximum insert plate thickness is the same as the stated maximum shell plate thickness for the Design Metal Temperature and corresponding Group for each grade.
† Group 1—Material as rolled, fully killed, fine grain.
‡ Group 2—Normalized, fully killed, fine grain.
§ Group 3—Quenched and tempered, fully killed, fine grain.

Table 29 Category 3 material requirements

Specification	Grade	Maximum Thickness *in.*	*(mm)*
ASTM A517/A517M	A, B	1¼	(32)
	E, F, H	1½	(38)

14.2.4 *Design metal temperature.* Unless otherwise specified, the design metal temperature (DMT) shall be taken as the lowest one-day mean ambient temperature for the location of the tank (see Figure 6) plus 15°F (–9.4°C).

14.2.5 *Low-hydrogen electrodes.* Low-hydrogen electrodes shall be used for all shielded metal-arc welding of shell courses having a thickness greater than ½ in. (13 mm) and DMT below 20°F (–6.7°C), including the welding of the shell-to-bottom joint of such shell courses.

14.2.6 *Other material requirements.*

14.2.6.1 Plate necks, reinforcing plates, or insert reinforcement of penetrations shall be of a material suitable for the thickness and temperature range,

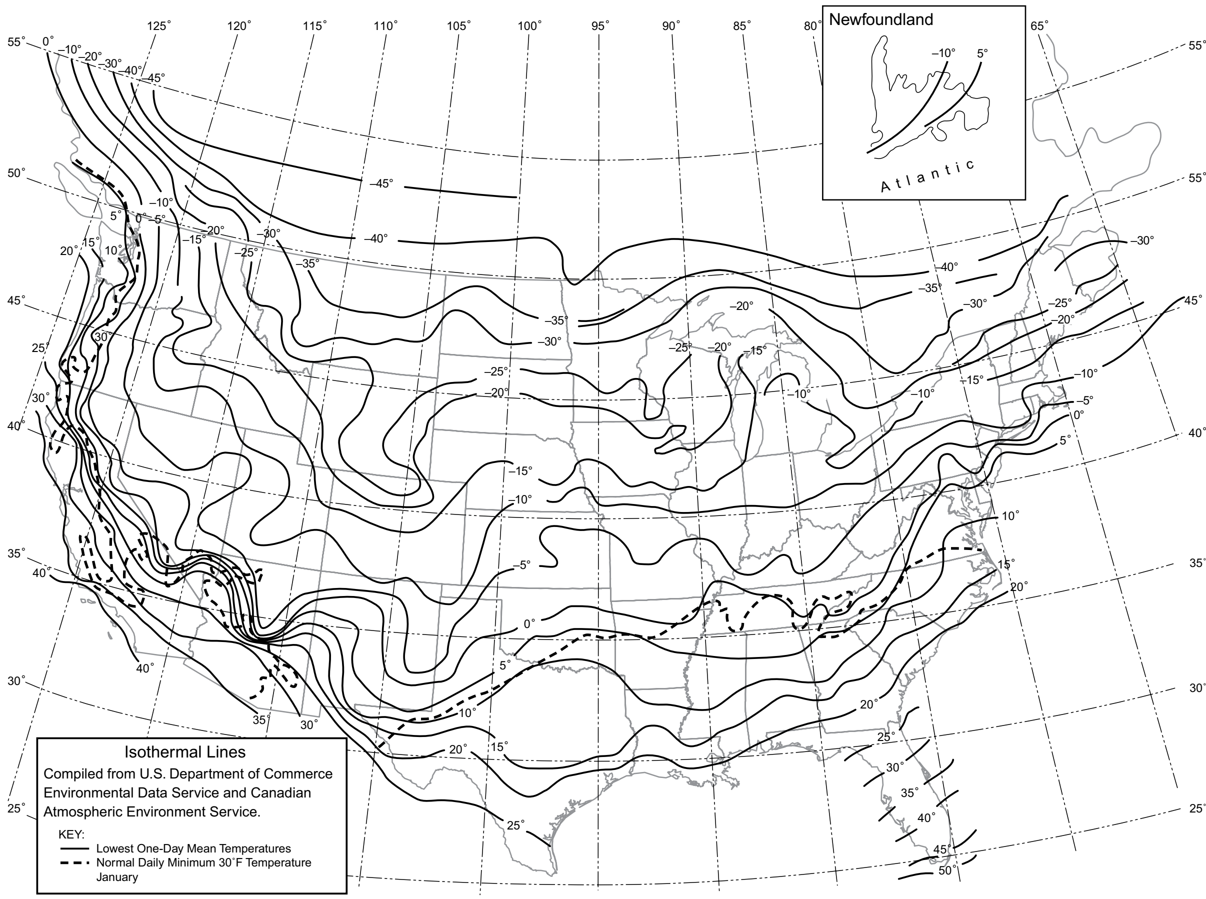


Figure 6 Isothermal lines for lowest one-day mean temperatures and normal daily minimum 30°F (–1.1°C) temperature line for January, United States and southern Canada

as selected from the appropriate category. Only materials with a design allowable tensile stress equal to or greater than the design allowable tensile stress in the shell shall be used as insert plates. Lower strength material may be used for reinforcement, provided that the area of reinforcement is increased in inverse proportion to the ratio of the allowable stress value of the reinforcement. No credit may be taken for the additional strength of any reinforcement having a higher allowable stress than that of the shell.

14.2.6.2 Continuous circumferential members such as stiffeners, balconies, or wind girders shall not be located within 6 in. (152 mm) of a horizontal shell joint. Continuous fillet welds attaching these members may cross vertical shell joints provided that (a) the angle of intersection of the two welds is approximately 90°, and (b) any splice welds in the attachment are at least 6 in. (152 mm) away from the vertical shell joints.

14.2.6.3 Permanent attachments to the shell, including stairways and similar low-load appurtenances, such as rafter clips, ladder clips, painters' angles, nameplates, insulation supports, and stiffening for roof knuckle, may be made of certified or noncertified* material and may be attached to shell courses designed in accordance with the requirements of this section, provided that the details of these attachments conform to the following requirements and that consideration is given to the movement of the shell (particularly movement of the bottom course) under hydrostatic loading:

1. Before the hydrotest, permanent attachments may be welded directly to the shell with fillet welds having a maximum leg dimension of ½ in. (13 mm) or less. The edge of any permanent attachment welds shall not be closer than 3 in. (76 mm) from the horizontal joints of the shell nor closer than 6 in. (152 mm) from the vertical joints, insert plate joints, or reinforcing plate fillet welds. Shell courses are exempt from this requirement where the maximum tensile hoop stress at the point of attachment is less than one-third the design allowable tensile stress.
2. Electrode requirements for welding permanent attachments to shell courses of category 1 material shall be the same as for the shell plate joint welds of the shell courses to which the attachments are welded (see Sec. 14.2.5).
3. Permanent attachments to category 2 and 3 shell courses shall be welded with low-hydrogen electrodes.
4. Welds for permanent attachments shall be inspected for cracks as follows. Identified cracks shall be repaired.
   a. Welds attaching the top angle, wind girder, intermediate stiffeners, and low-load appurtenances are exempt from magnetic-particle or dye-penetrant inspection when attached to shell courses with a minimum specified tensile strength of less than 75,000 psi (517.1 MPa).
   b. Inspection of pilaster welds to shell courses of category 1 material shall be by magnetic particle or dye-penetrant methods, tested for a minimum distance of 6 in. (152 mm) on either side of the girth joints crossed by the pilasters.
   c. Other permanent attachment welds shall be fully inspected by either the magnetic-particle or dye-penetrant method. Inspection of attachment welds to category 3 shell material shall not commence until 48 h after welding has been completed.

* Noncertified materials are materials that are identified as meeting one of the acceptable material specifications of this section, but for which no certified mill test reports are available.

5. In addition to other essential variables, preheat requirements for thick plates or low-atmospheric temperature during welding shall be specified in the selected welding procedure.

6. Temporary attachments to shell courses of any Sec. 14 material described in this section shall be made prior to the hydrostatic test. Weld spacing for temporary attachments made after welding of the shell joints shall be the same as that required for permanent attachments. Temporary attachments to Sec. 14 shell courses shall be removed, and any resulting damage shall be repaired and ground to a smooth profile prior to the hydrostatic test.

7. The material is identified and is suitable for welding.

8. The material is compatible, insofar as welding is concerned, with that to which the attachment is to be made.

9. For attachments to quenched and tempered shell plates, the permanent attachment shall be made of material whose published minimum yield point (strength) is not less than 1.5 times the design tensile stress of the shell plate.

14.2.6.4 Attachment details for insert-type fittings. Details for attaching insert-type fittings to shells constructed with ASTM A517/A517M material shall conform to ASME BPVC Sec. VIII, Div. 1, paragraph UHT-18 and Figure UHT 18.1.

14.2.6.5 Pipe reinforcing. Pipe conforming to ASTM A53/A53M, Type E or S, Grade B, or API 5L, Grade B, may only be used for penetrations through category 1 or 2 shell materials for design metal temperatures +20°F (–6.7°C) or warmer. Pipe conforming to ASTM A106/A106M, Grade B, may only be used for penetrations through category 1 or 2 shell materials for design metal temperatures +5°F (–15°C) or warmer. Pipe conforming to ASTM A333/A333M, Grade 6, or ASTM A524 may be used for penetrations through category 1 or 2 shell materials for temperatures –50°F (–45.6°C) or warmer. No pipe or fabricated penetrations shall be used in shell courses constructed from category 3 materials (see Sec. 14.2.6.7 and Sec. 14.3.2.5).

14.2.6.6 Reinforcing. Reinforcing is not required around openings 2 in. (51 mm) in diameter or less. Reinforcing around openings greater than 2 in. (51 mm) in diameter shall conform to the requirements of Sec. 3.13. In calculating reinforcing, the hole shall be considered equal to the outside diameter of the pipe. Pipe material within the limits of reinforcing may be counted as reinforcement, provided that such area is reduced by the ratio of published minimum yield point (strength) of the pipe to published minimum yield point (strength) of the shell.

If the published minimum yield point (strength) of the pipe is higher than the published minimum yield point (strength) of the plate, no increase of such area shall be allowed.

14.2.6.7 Nozzle forgings. Nozzle forgings conforming to ASTM A105/A105M or ASTM A181/A181M, Class 70, may be used with category 1 and 2 materials for design metal temperatures 0°F (–17.8°C) and higher. Nozzle forgings used with category 1 and 2 shell materials for design metal temperatures below 0°F (–17.8°C) shall conform to ASTM A350/A350M LF1 or LF2.

For all penetrations in shell courses constructed with category 3 material, only ASTM A592/A592M forgings meeting the same toughness requirements as required for the shell plates shall be used.

Sec. 14.3 General Design

14.3.1 *Joints.*

14.3.1.1 Shell joints. Vertical and horizontal shell plate joints, except shell-to-bottom annulus joints and shell-to-top angle joints, shall be double-welded butt joints with complete joint penetration. The top angle may be attached to the shell by a double-welded butt joint with complete joint penetration or double-welded lap joint with fillet welds. Shell-to-bottom annulus joints shall be continuous fillet-welded joints on both sides of the shell in accordance with Sec. 8.7.

14.3.1.2 Joint efficiency. The joint efficiency for double-welded butt joints shall be 100 percent.

14.3.2 *Shells.*

14.3.2.1 Allowable tensile stress. The maximum design allowable tensile stress in tension for shell plates shall be no greater than 60 percent of the published minimum yield point (strength) or one-third of the published minimum tensile strength, whichever is smaller. Table 30 provides a summary of acceptable steels and allowable stresses. A one-third increase in design allowable tensile stress for shell plates is permitted for load combinations that include seismic loads.

14.3.2.2 Analysis. The shell membrane hoop stress may be computed by the formula in Sec. 3.7 or by shell analysis theory.[†] Boundary conditions for shell analysis theory shall assume a fully plastic moment in a ¼-in. (6.35-mm) bottom plate thickness, regardless of the actual bottom plate thickness required, and zero radial displacement deflection. For tanks with a height-to-diameter (H/D) ratio of 0.50 or less, the Variable Design Point Method in API 650 may be used. Design allowable tensile stresses shall be in compliance with Sec. 14.3.2.1.

[†] Shell analysis theory may be ANSYS, Kalmin, STRUDL, or other commercially available finite element program.

Table 30 Maximum design tensile stresses in shell plates in contact with water

Specification	Specification Title	Grade	P No.	Maximum Design Tensile Stress psi	(MPa)
ASTM A36/A36M	Specification for Carbon Structural Steel		1	19,330	133.3
ASTM A131/A131M	Specification for Structural Steel for Ships	A, B	1	19,330	133.3
ASTM A516/A516M	Specification for Pressure Vessel Plates Carbon Steel, for Moderate and Lower Temperature Service	60	1	19,200	132.4
		70	1	22,800	157.2
CSA G40.21	Structural Quality Steel	38W, 38WT	1	20,000	137.9
		44W, 44WT	1	21,670	149.4
ASTM A283/A283M	Specification for Low and Intermediate Tensile Strength Carbon Steel Plates	C	1	18,000	124.1
ASTM A662/A662M	Specification for Pressure Vessel Plates, Carbon Manganese, or Moderate and Lower Temperature Service	B	1	21,670	149.4
		C	1	23,330	160.9
ASTM A573/A573M	Specification for Structural Carbon Steel Plates of Improved Toughness	58	1	19,200	132.4
		70	1	23,330	160.9
ASTM A588/A588M	Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi (345 MPa) Minimum Yield Point, with Atmospheric Corrosion Resistance	All	1	23,330	160.9
ASTM A633/A633M	Specification for Normalized High Strength Low Alloy Structural Steel Plates	C, D	1	23,330	160.9
ASTM A537/A537M	Specification for Pressure Vessel Plates, Heat Treated, Carbon Manganese Silicon Steel	Cl 1	1	23,330	160.9
		Cl 2	1	26,670	183.9
ASTM A517/A517M	Specification for Pressure Vessel Plates, Alloy Steel, High Strength, Quenched and Tempered	A, B, E, F, H	11B	38,330	264.3

14.3.2.3 Maximum plate thickness. The maximum thickness of any shell plate at the welded joint shall be 1½ in. (38 mm). Insert plates more than 1½ in. (38 mm), up to the maximum thickness limit listed in Tables 27 and 28 for the material grade and design metal temperature, may be used, provided that the base metal is impact tested to the requirements of Sec. 14.1.5 and the following special requirements are met: insert plates shall have 5 ft-lbf (6.8 N-m) added to the transverse impact requirements of Sec. 14.1.4 and Sec. 14.1.5 for each ¼ in. (6.35 mm), or fraction thereof, that their thickness exceeds 1½ in. (38 mm). See Table 27 for exemption of impact testing of insert plates.

14.3.2.4 Upper-shell courses. In the interest of economy, upper courses may be of weaker material than used in the lower courses of shell plates, but in no case shall the calculated stress of any course be greater than permitted for the material in that course. A plate course may be thicker than the course below it, provided that the extra thickness is not used in any stress or wind stability calculation.

14.3.2.5 Penetrations in category 3 material. When the shell plates are a category 3 material, manholes, other penetrations, and their reinforcing plates shall not be attached to the shell with fillet welds. Only forged insert-type fittings welded to the shell with butt joints shall be used, and fitting-to-shell welds shall be completely radiographed (see Sec. 14.2.6.4 through Sec. 14.2.6.7).

14.3.2.6 Penetrations. Penetrations that are 12 in. (300 mm) or greater in diameter through a shell plate thicker than 1 in. (25 mm) shall be prefabricated into a shell plate and thermally stress relieved.

14.3.2.7 Weld clearance above bottom. The weld around a penetration, the butt-joint weld around the periphery of an insert plate, or the fillet weld around the periphery of a reinforcing plate for openings of all sizes shall be at a distance above the bottom of the tank of at least 10 times the shell thickness or 12 in. (300 mm), whichever is greater. Insert plates or reinforcing plates extended to the bottom shall be permitted as an alternative, provided that the low point of the cutout above the bottom is at least 10 times the shell thickness or 12 in. (300 mm), whichever is greater.

14.3.2.8 Weld spacing. The spacing of welds around penetrations, reinforcing plates for penetrations, and thickened insert plates shall conform to the following:

14.3.2.8.1 The outer weld toe of a nonstress-relieved weld around the periphery of a penetration, around the periphery of a thickened insert plate, or around the periphery of a penetration reinforcing plate shall be at least the greater of eight times the weld size or 10 in. (250 mm) from the centerline of any butt joints in the shell.

14.3.2.8.2 Where stress relieving of the periphery weld has been performed before welding of the adjacent shell joint, the spacing may be reduced to 6 in. (150 mm) from vertical joints or 3 in. (76 mm) from horizontal joints, provided that, in either case, the spacing is not less than 2½ times the shell thickness.

14.3.2.9 Bottom annulus. For all tank shells using ASTM A517/A517M steel, tank shells designed to 26,000 psi (179.3 MPa) or higher, and lower-stressed tank shells greater than 150 ft (45.7 m) in diameter, bottom annulus with butt joints shall project at least 24 in. (610 mm) from the inside of the shell to any lapped bottom plates. This bottom annulus may be provided by butt-joint welding the usual sketch plates or a separate ring of annular plates. If the bottom annulus is provided by butt-joint welding of the sketch plates, sketch plate joints shall be butt-joint welded their full length. Single-welded bottom annulus radial joints shall be made using a backup bar and a root opening in the joint to ensure complete joint penetration and fusion into the backup bar. The bottom annulus to which the shell attaches shall be of material specified in category 1 or 2 for the thickness and design metal temperature. For self-anchored tanks, where a butt-joint welded bottom annulus is required by this section, at least 2 in. (51 mm) of the bottom shall project outside the shell. The thickness of the bottom annulus shall be a function of the stress level and bottom course thickness, *t*. Refer to Table 31.

Sec. 14.4 Inspection

When magnetic-particle or liquid-penetrant testing is required, the personnel performing such examination or testing shall be qualified in accordance with ASNT SNT-TC-1A or shall be certified as competent in accordance with the rules of ASME BPVC Sec. VIII, Div. 1, Appendix 6 or 8, as applicable.

When the shell plates are of quenched and tempered steels with a published minimum tensile strength of 75,000 psi (517.1 MPa) or greater, welds in the shell shall be inspected on both sides by the magnetic-particle method. Regardless of the shell material used, all welds attaching manholes, nozzles, and other penetrations shall be inspected for cracks by either the magnetic-particle or the dye-penetrant method.

Table 31 Minimum thickness of bottom annulus

Maximum Design Stress Level of Bottom Shell Course	*psi*	*(MPa)*	Minimum Thickness of Bottom Annulus *in. (mm)*
Less than	26,000	(179.3)	¼ (6.4)
Greater than 26,000 psi and less than	28,500	(196.5)	0.1875*t**
ASTM A517/A517M	38,330	(264.3)	0.50*t**

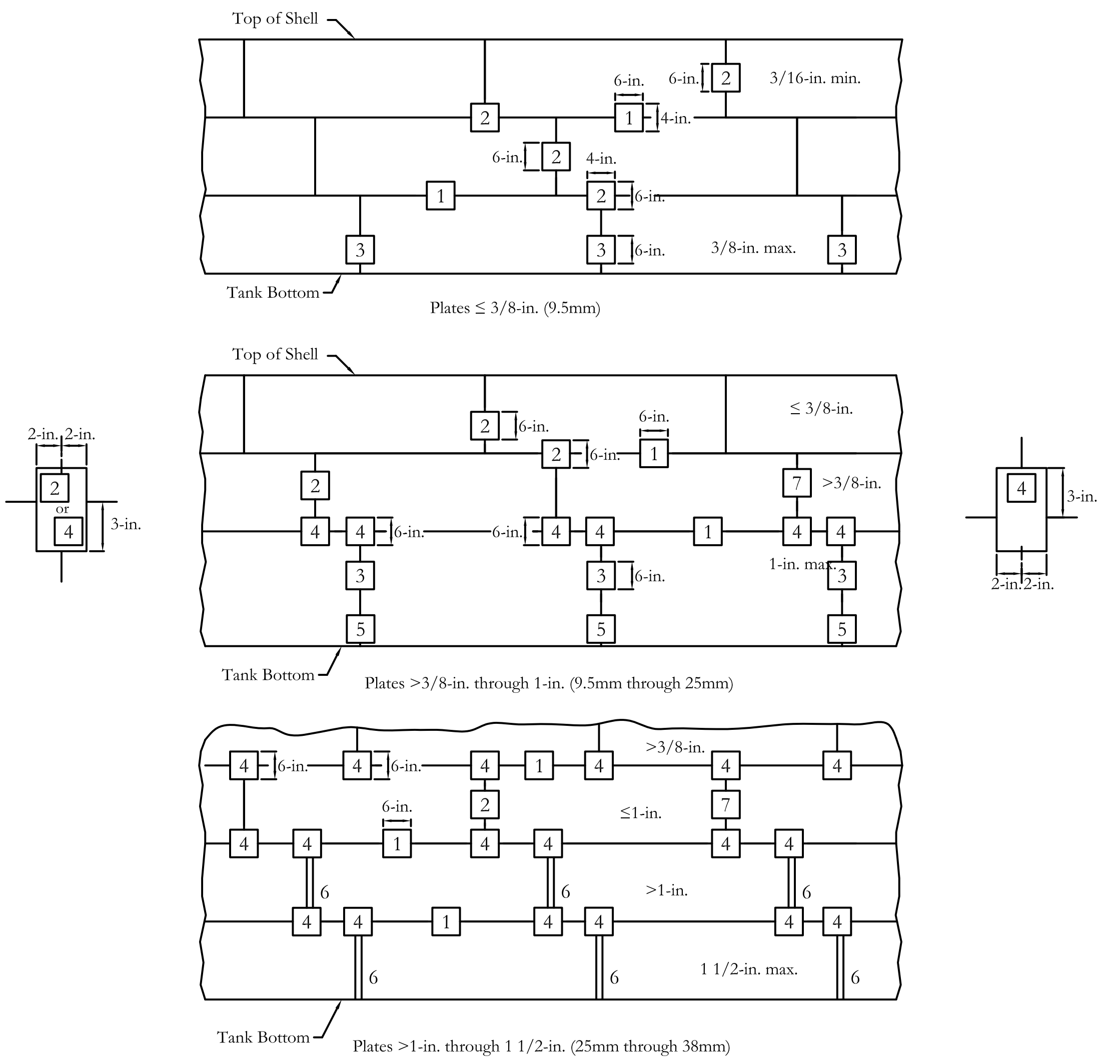
* *t* is the bottom course thickness. In no case is the bottom annulus to be less than ¼ in. (6.4 mm) thick.

14.4.1 *Fillet welds.* Fillet welds shall be inspected visually when not otherwise specified in this section.

14.4.2 *Shell-to-bottom joint.* See Sec. 11.10.2 for leak testing of the bottom-to-shell joint.

14.4.3 *Butt joints.* Radiographic image quality for each thickness shall conform to Sec. 11.6.2. All butt joints in the shell, both vertical and horizontal, shall be radiographed in accordance with Sec. 11.5, except for Sec. 11.5.1 and Sec. 11.5.2, which shall be superseded by the following rules for the number, size, and location of radiographs:

1. Butt joints in shell plates, either plate of which has a thickness of ⅜ in. (9.5 mm) or less:
   a. One radiograph shall be taken in the first 10 ft (3 m) of completed vertical joint and horizontal joint of each type and thickness welded by each welder or welding operator.
   b. Thereafter, without regard to the number of welders or welding operators working thereon, one additional radiograph shall be taken in each 100 ft (30.5 m) of vertical joint and 100 ft (30.5 m) of horizontal joint, and any remaining major fraction thereof of the same type and thickness joint.
   c. At least 25 percent of the selected vertical spots, with a minimum of two per tank, shall be at junctions of vertical and horizontal welds.
   d. In addition to the foregoing requirement, one random radiograph shall be taken in each vertical joint in the lowest course.
2. Butt joints in which the thickness of the thinner plate is more than ⅜ in. (9.5 mm), but not thicker than 1 in. (25 mm), shall be radiographed in accordance with Sec. 14.4.3, rule 1, with the following exceptions:
   a. Junctions of vertical and horizontal seams shall be radiographed.
   b. In the lowest course, two radiographs shall be taken in each vertical joint, one of which shall be taken as close to the bottom as practical; the second shall be taken at random.
   c. Above the lowest course, one radiograph shall be taken in 25 percent of the vertical joints for shell rings that exceed ¾ in. (19 mm) in thickness.
3. Butt joints in which the thickness of the thinner plate is thicker than 1 in. (25 mm) shall be radiographed in accordance with Sec. 14.4.3, rule 1, with the following exceptions:
   a. Vertical butt joints shall be fully radiographed.
   b. Junctions of vertical and horizontal joints shall be radiographed.



Numbers in the figure illustrate the following requirements:	Section No.
(1) horizontal radiograph, one in first 10 ft (3 m) for same type and thickness of joint for each welder or welding operator - thereafter, each 100 ft (30 m) of joint or remaining major fraction thereof;	14.4.3. (1) a., b. and c.
(2) vertical radiograph, one in first 10 ft (3 m) for same type and thickness of joint for each welder or welding operator - thereafter, each 100 ft (30 m) or remaining major fraction thereof, 25 percent of which are to be intersections with a minimum of 2 intersections per tank;	14.4.3 (1) a., b. and c.
(3) random vertical radiograph in each vertical seam in lowest course;	14.4.3 (1) d.
(4) radiograph at all intersections of vertical with horizontal seams;	14.4.3 (2) a.
(5) additional radiograph near the bottom of each vertical seam in lowest course;	14.4.3 (2) b.
(6) vertical seams fully radiographed;	14.4.3 (3) a.
(7) when shell rings above the bottom course exceed 3/4-in. (19 mm) in thickness, at least one radiograph in 25 percent of vertical seams of shel rings more than 3/4-in. (19 mm thick).	14.4.3 (2) c.

Figure 7 Radiographic requirements for tank shells according to Sec. 14

4. Figure 7 illustrates the radiographic requirements specified in this section and is included for clarity.

5. Plates shall be considered the same thickness when the thickness difference is ⅛ in. (3 mm) or less.

14.4.4 *Inspection of butt joints in annular plates.* When a butt-joint welded bottom annulus is required by Sec. 14, the radial joints shall be inspected according to the criteria in Sec. 14.4.4.1 and Sec. 14.4.4.2.

14.4.4.1 Double-welded joints. Double-welded radial joints shall be 100 percent visually inspected. In addition, 10 percent of the radial joints shall be inspected by taking one spot radiograph at least 6 in. (152 mm) in length as close to the outer edge as possible.

14.4.4.2 Single-welded joints. Single-welded radial joint shall be 100 percent visually inspected and, in addition, shall be inspected by one of the following methods:

1. One spot radiograph at least 6 in. (152 mm) in length shall be taken as close to the outer edge as possible. If the radial joint is not completed at one time, a second radiograph or inspection by air carbon arc gouging (Sec. 11.8) shall be made on the second portion welded.
2. A localized weld inspection shall be made using air carbon arc gouging (Sec. 11.8) as close to the outer edge as possible. If the radial joint is not completed at one time, a second localized weld inspection using air carbon arc gouging shall be made on the second portion welded.

14.4.5 *Welding inspector.* A certified welding inspector who shall be responsible for all weld inspections in accordance with this standard may be specified for a tank that is to comply with this section. The welding inspector shall have prior tank-inspection experience and shall be a certified welding inspector in accordance with the provision of AWS QC1.

Sec. 14.5 Certification of Compliance

A certification, in the form of Figure 8, shall be provided stating that the tank has been designed, fabricated, erected, inspected, and tested in accordance with Sec. 14 and that all such inspection and testing has been satisfactory.

CERTIFICATION OF COMPLIANCE WITH AWWA REQUIREMENTS OF ANSI/AWWA D100, SECTION 14

To ____
(Name and address of purchaser or owner)

____

We hereby certify that the ______________________________________
(Standpipe or Reservoir)

constructed for you at ______________________________________
(Location of standpipe or reservoir)

and described as follows: ______________________________________
(Dimensions, capacity, and type of structure)

____

has been designed, fabricated, erected, tested, and inspected in accordance with all of the requirements of Section 14 of ANSI/AWWA D100-___, entitled "Welded Carbon Steel Tanks for Water Storage" and that the results of all inspections, radiographs, and other tests indicate that the standpipe or reservoir fully complies with the requirements of Section 14.

____
(Name of company)

____
(Authorized representative)

____
(Date)

County of ______________________

State of ______________________

Acknowledged and sworn to before me

This _______ day of ______________________, 20____

______________________ Notary Public

Figure 8 Certification of compliance with requirements of ANSI/AWWA D100, Sec. 14

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APPENDIX A

Commentary for Welded Carbon Steel Tanks for Water Storage

This appendix is for information only and is not a part of ANSI/AWWA D100. Numbers following "A" reference the applicable section in the body of the standard.*

SECTION A.1: GENERAL (REFER TO SEC. 1 OF ANSI/AWWA D100.)

Sec. A.1.1 Scope

A.1.1.2 *Items not covered.* This standard does not address painting and disinfecting tanks. Refer to ANSI/AWWA D102, Coating Steel Water-Storage Tanks, for painting requirements and ANSI/AWWA C652, Disinfection of Water-Storage Facilities, for disinfection requirements. If the tank is to be painted and disinfected, the following sequence may be used:

1. Complete inside tank painting.
2. Provide a minimum drying time corresponding with the paint system used. The minimum drying time under any paint system should not be less than five days after the final inside coat is applied.
3. Disinfect the tank.
4. Fill the tank with potable water and test.
5. Place the tank into service. The outside paint system need not be complete if temperature conditions prevent completion.

Regardless of the sequence used for painting and testing the tank, it should be disinfected after the final inside paint coat has dried in accordance with the requirement for the paint used, and the tank may be filled with potable water and placed into service.

Sec. A.1.4 References

The requirements contained in the references listed in Sec. 1.4 are part of the standard. The following references contain useful information related to steel tanks for water storage:

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

1. ACI[†] 355.3R-11—Guide for Design of Anchorage to Concrete: Examples Using ACI 318 Appendix D.

2. API[‡] 653—Tank Inspection, Repair, Alteration, and Reconstruction, 4th Edition, Addendum 3 November 2013.

3. AWS[§] Z49.1-12—Safety in Welding, Cutting and Allied Processes.

4. ASME[¶] BPVC-CC-BPV—BPVC Code Cases, 2013 Edition.

5. ASME STS-1-2011—Steel Stacks.

6. AWWA Manual M42—*Steel Water-Storage Tanks,* Revised Edition (2013).

7. NFPA** 22-13—Standard for Water Tanks for Private Fire Protection.

8. NFPA 51B-14—Standard for Fire Prevention During Welding, Cutting, and Other Hot Work.

9. Technical Information Document 7024. 1963. *Nuclear Reactors and Earthquakes*. Chapter 6 and Appendix F. Published by Lockheed Aircraft Corporation under a grant from the US Department of Energy (formerly US Atomic Energy Commission).

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

11. Baker, E.H., et al. *Shell Analysis Manual*. NASA-CR-912. Washington, D.C.: National Aeronautics and Space Administration.

12. Baker, E.H., L. Kovalevsky, and F.L. Rish. 1972. *Structural Analysis of Shells*. New York, N.Y.: McGraw-Hill.

13. Housner, G.W. 1954. *Earthquake Pressures on Fluid Containers*. California Institute of Technology.

14. Malhotra, P.K., T. Wenk, and M. Wieland. *Simple Procedure for Seismic Analysis of Liquid-Storage Tanks*. Structural Engineering International, 3/2000.

15. Miller, C.D., S.W. Meier, and W.J. Czaska, 1997. Effects of Internal Pressure on Axial Compressive Strength of Cylinders and Cones, Structural Stability Research Council.

[†] American Concrete Institute, 38800 Country Club Drive, Farmington Hills, MI 48331.

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

[§] American Welding Society, 550 N.W. LeJeune Road, Miami, FL 33126

[¶] ASME International, Two Park Avenue, New York, NY 10016.

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

16. Veletsos, A.S. 1984. *Seismic Response and Design of Liquid Storage Tanks*. Guidelines for the Seismic Design of Oil and Gas Pipeline Systems, ASCE, New York: 255–370.

17. Veletsos, A.S., and J.Y. Yang. 1976. *Dynamics of Fixed-Base Liquid Storage Tanks*. Houston, Tex.: Rice University.

18. Wozniak, R.S., and W.W. Mitchell. 1978. *Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks*. 1978 Proceedings—Refining Department, Washington, D.C.: American Petroleum Institute. 57:485–501.

SECTION A.2: MATERIALS (REFER TO SEC. 2 OF ANSI/AWWA D100)

Sec. A.2.2 Material Requirements

A.2.2.1 *Bolts, anchor bolts, and rods.* Requirements for nuts are found in the ASTM bolt specifications. For ASTM A36/A36M rods, the nut requirements of ASTM A194/A194M typically apply.

SECTION A.3: GENERAL DESIGN (REFER TO SEC. 3 OF ANSI/AWWA D100)

Sec. A.3.1 Design Loads

In preparing this standard, the Revision Task Force and Committee primarily used ASCE 7-10 and subsequent revisions as the basis for the minimum design loads.

A.3.1.1 *Risk Category.* Under ASCE 7-10, the assignment of a Risk Category was introduced, replacing the traditional Occupancy Category. That assignment of risk category remains consistent in ASCE 7-16. The risk category is used in the selection of importance factors for snow load and seismic load. The risk category must be considered when referencing a map for selection of basic wind speed.

Selection of a risk category is based on tank function and potential 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 environmental loadings and indirectly influences the performance level expected 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. Table A.1 can be used to convert Occupancy Category to Risk Category for use with this standard.

A.3.1.1.1 Risk Category II. Risk Category II is used for tanks not designated Risk Category III or IV.

A.3.1.1.2 Risk Category III. 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.

A.3.1.1.3 Risk Category IV. 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 back-up 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).

Table A.1 Risk category

Occupancy Category	Risk Category		
	II	**III**	**IV**
I	X		
II	X		
III		X	
IV			X

A.3.1.4.1 Ground snow load. At sites falling within a mapped area labeled CS or at elevation above the limit identified in ASCE 7, Fig. 7-1, the ground snow load may be highly variable within a small geographical region. The determination is required to be based on the statistical analysis described. The requirement for the purchaser to provide this ground snow load value is based on the dependence of the analysis on local records and experience. Additional information may be available from the local authority having jurisdiction.

A.3.1.6 *Wind load.* Wind loads calculated using ASCE equations are strength level loads. In Eq 3-4, a scaling factor is included to bring the velocity pressure from strength level to service level. In Eq 3-5, 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 D100.

A.3.1.6.1 Basic wind speed. Basic wind speed is selected from one of three maps dependent upon the assigned risk category. By using maps and basic wind speeds according to risk category, the importance factor, I, is no longer required within the wind pressure equations. The maps are included in ANSI/AWWA D100 by reference only to ASCE 7. Basic wind speed values may also be obtained from web-based services such as the ASCE 7 Hazard Tool and the ATC Hazards by Location tool.

A.3.1.6.3 Topographic effects. ASCE 7 includes a topographic factor (K_{zT}) to account for wind escalation over hills, ridges, and escarpments. ANSI/AWWA D100 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}.

A.3.1.6.4 Gust effects. The gust-effect factor, G, accounts for along-wind loading effects caused by wind turbulence–structure interaction. This standard uses a gust-effect factor of 0.85 for tanks that can be classified as rigid structures, such as single-pedestal tanks with large pedestal diameters. The gust-effect factor for tanks classified as flexible structures shall be evaluated per ASCE 7 guidelines but shall not be less than 0.85.

A.3.1.6.5 Velocity pressure. ANSI/AWWA D100 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 versions of ASCE 7 before 2005 included a directionality factor of 0.85 within the load factor for wind for ultimate strength design. ASCE 7-16 considers that K_d = 1.0 for load factor purposes, and the calculated wind loads may be used with load factors that have not been adjusted upward to account for the removal of this factor.

A.3.1.6.7 Wind load, *W.* The design wind pressure is applied to the projected area of the structure within each height zone and change in structure shape as shown in Figure A.1. The wind shear at the base of the structure is then the summation of wind loads from each height zone. The wind moment at the base of the structure is then the summation of wind loads multiplied by the moment arms of each height zone. The moment arm is the distance from the structure base to the centroid of the wind load projected area for each height zone.

A.3.1.6.7.3 Wind-structure interaction. Reference 5 of Sec. A.1.4 is one source for checking dynamic wind effects on slender single-pedestal tanks and slender standpipes.

A.3.1.8 *Balcony, platform, ladder, stair, and localized roof loads*. Ladder loads used for design shall be based on maximum intended load per OSHA regulations. The loads of ASCE 7 shall be considered minimum values. The purchaser should also consider other conditions that may require specification of loads greater than minimum. Such conditions may include rescue operations, equipment

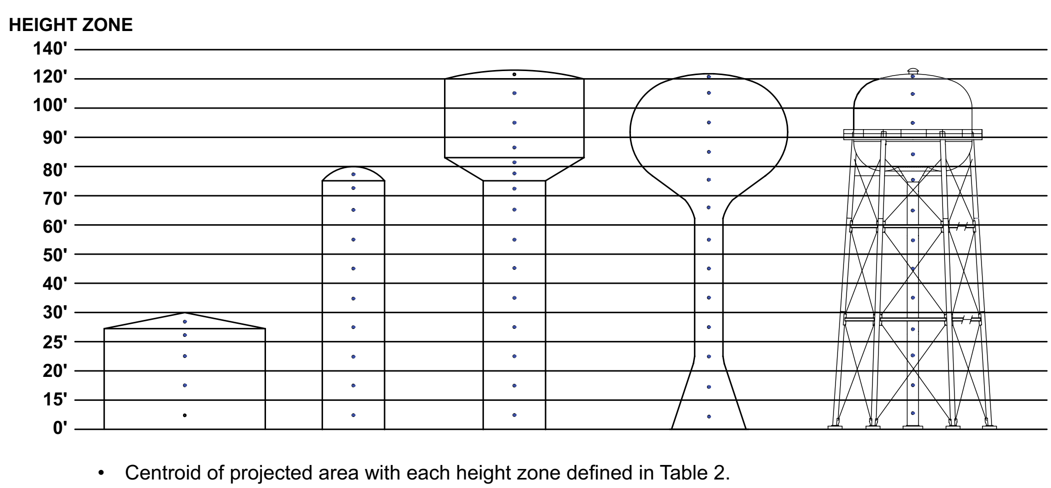


Figure A.1 Application of height zones for design wind pressures attachment, maintenance activity, and other conditions. The purchaser may specify corrosion allowance to be applied to ladder components.

Sec. A.3.2 Unit Stresses

A.3.2.2 *Pipe thickness underrun.* Some pipe specifications allow thickness underruns as high as 12.5 percent. The appropriate specification shall be consulted for allowable underrun to determine what adjustments in thickness need to be made to ensure that minimum design thicknesses are met.

Sec. A.3.4 Allowable Compressive Stresses for Columns, Struts, and Shells

A.3.4.3 *Double-curved axisymmetrical, conical, and cylindrical sections.* The provisions for shell stability apply to shells subjected to uniaxial compression or biaxial compression–tension (circumferential membrane stress is zero or tension). Code Case 2286 of reference 4 of Sec. A.1.4 may be used to design shells subjected to biaxial compression or biaxial compression–tension (circumferential membrane stress is compressive).

Two methods are now given to determine the allowable local buckling compressive stress, F_L. Alternative procedure, Method 2, from previous versions of ANSI/AWWA D100 is deleted, and Method 3 is renamed Method 2. Alternative procedure, Method 2, is based on analysis and includes the stabilizing effect of hydrostatic pressure. Method 2 only applies to water-filled shells having radius-to-thickness ratios less than 1,000 and greater than or equal to the value of $(R/t)_c$ given in Table 10. The 1,000 limit represents a practical upper limit. The stabilizing effect of hydrostatic pressure has little effect when the radius-to-thickness ratio is less than $(R/t)_c$.

Experimental investigations (see reference 15 of Sec. A.1.4) have shown that the critical buckling stresses of axially loaded cylinders and cones are significantly increased by internal pressure for cylinders and cones that would fail elastically under axial load alone. The destabilizing effect of initial imperfections is reduced. The circumferential tensile stress induced by pressurization inhibits the diamond-shaped buckling pattern associated with buckling under axial load only, and as the circumferential stress is increased, the buckling mode of the cylinder or cone will change to the axisymmetric mode. For shells constructed of materials that may fail elastically, such as mylar or metals with a high yield stress, the buckling stress may reach the classical buckling stress. For steels with lower yield stresses, the buckling stress is limited by material failure. A modified von Mises failure theory has been found to define this limit. Pressure stability has little effect on the buckling stresses of steel cylinders and cones with *R/t* ratios less than about 300.

The elastic buckling stresses of shells without pressurization are influenced by the magnitude of initial imperfections. The amplitudes, *e*, of the imperfections are measured from a straight template held anywhere against any meridian. The length of the template, L_x, equals $4\sqrt{Rt}$ and is approximately equal to one half wavelength of the buckles that form in a cone or cylinder under axial compression. The buckle form will be sinusoidal in shape even though the shell has irregular contours. The buckling load is a function of the amplitude of the prescribed imperfection. The most critical amplitude is the value measured over half the wavelength. The buckling load at any elevation is a function of the maximum value of *e* at that elevation. Therefore, the imperfections located $2\sqrt{Rt}$ above or below the elevation being investigated will not affect the buckling load. When the values of *e* are less than or equal to $0.04\sqrt{Rt}$, the elastic buckling coefficient, C_o, given by Eq 3-16 applies. When *e* equals $0.08\sqrt{Rt}$, the value of C_o is halved. Linear interpolation applies to values of *e* between $0.04\sqrt{Rt}$ and $0.08\sqrt{Rt}$. For cones, L_x is based on the value of *R* at the midpoint of the template.

Unlike the coefficient C_o, the magnitude of initial imperfections has little effect on the elastic buckling coefficient for pressure stabilization, C_p. Eqs 3-19 and 3-20 can be used to determine C_p for shells with values of *e* less than or equal to $0.08\sqrt{Rt}$.

A.3.4.3.1 Method 1. Method 1 is mandatory for shells that do not contain water and for ground-supported, flat-bottom tanks.

A.3.4.3.2 Method 2. Method 2 equations are based on shells with maximum deviation, *e*, equal to $0.04\sqrt{Rt}$. A nonlinear buckling analysis is required to determine the critical buckling stress, F_{cr}. The standard provides minimum requirements for determining the critical buckling stress. The critical buckling strength is very sensitive to the magnitude, shape, and length of the assumed imperfection. A more severe or larger imperfection than specified, or a smaller gauge length with a proportionally smaller deviation, should be used if appropriate for the method of construction.

Eq 3-17 for the allowable local buckling compressive stress, F_L, includes a factor of safety of 2. The allowable local buckling compressive stress is limited to the critical buckling stress from reference 15 of Sec. A.1.4 (Eq 3-17), divided by a factor of safety of 2.

Sec. A.3.5 Shell Girder, Intermediate Stiffeners, and Compression Rings

A.3.5.1 *Top shell girders.* Refer to API 650 for details related to top and intermediate shell girders.

A.3.5.2 *Intermediate shell girders.* Refer to API 650 for details related to top and intermediate shell girders.

Sec. A.3.6 Roofs

A.3.6.1 *General requirements.* 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 because of the differential settlements. Some possible solutions:

1. Correct or improve the foundation conditions 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.

A.3.6.1.5 Placement of rafters. During an overflow condition, the water level extends above the MWL. A minimum distance from the MWL to the bottom side of roof rafters or trusses may be specified.

A.3.6.1.7 Maximum rafter spacing. Eq 3-36 is empirical and is set to allow the historical 84-in. (2,135-mm) spacing on a 3⁄16-in. (4.76-mm) roof with a standard 25-psf (1,205 N/m^2) live load.

A.3.6.1.9 Supported-cone roofs. The roof plates for supported cone roofs are welded to each other and to the tank perimeter. Roof plates are not required to be welded to supporting members unless required by the structural design as part of the strength or bracing of the member. Therefore, the point of contact between the roof plates and supporting members does not constitute a joint unless the roof plate is welded to the supporting members. When welding of the roof plate to the supporting member is required by the structural design, the design calculations should specify the type, size, location, and spacing of the required welds.

Sec. A.3.8 Anchorage

A.3.8.1.2.2 The pullout capacity of the anchor shall be reduced when the horizontal radius of adjacent pullout cones overlap and when the free edge is less than the horizontal radius of the pullout cone from the anchor.

A.3.8.4 *Anchor requirements*. Anchors other than anchor bolts and anchor straps may be used, provided that they meet the following requirements:

1. When anchors are not exposed to weather, the minimum thickness of any portion of the anchor shall be ¼ in. (6.35 mm).
2. When anchors will be exposed to weather, a corrosion allowance of ⅜ in. (9.52 mm) shall be added to all portions of the anchor with a required design thickness less than ½ in. (12.7 mm), and a corrosion allowance of ¼ in. (6.35 mm) shall be added to all portions of the anchor with a required design thickness of ½ in. (12.7 mm) or greater.
3. The design of the attachment to the tank shall consider the anchor geometry, eccentricity, and weld configuration of the anchor attachment.

A.3.8.4.1 Anchor bolts. ACI 355.1R indicates that “J”- and “L”-type embedded anchor bolts are not recommended. The report states that bent, smooth, or deformed threaded bars have been known to straighten out in pull-out tests.

The allowable stresses for anchor bolts do not include allowance for prying action due to corner bearing on nuts. The placement tolerances of Sec. 12.4.7, Sec. 12.5.3, and Sec. 12.6.3 are intended to minimize such prying action.

A.3.8.6 *Design for resistance to base shear*. This standard provides requirements for uplift anchorage. 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.

Sec. A.3.9 Corrosion Allowance and Protection

A.3.9.1 *General*. Careful consideration shall be given to the proper allowance for corrosion. This allowance will depend on the corrosive nature of the stored water, the proximity of the tank to saltwater or other causes of atmospheric corrosion, and the care with which the paint or other protection will be maintained.

Sec. A.3.10 Minimum Thickness and Size

A.3.10.8 *Butt-welded annulus*. A butt-welded annulus may be provided for ground-supported flat-bottom tanks for one or more of the following reasons:

1. To resist seismic overturning moment for self-anchored tanks in accordance with Sec. 13.5.4.1.
2. To resist hoop tensile stress in accordance with Sec. 14.3.2.9.
3. To facilitate construction (i.e., construction ring).

SECTION A.4: SIZING AND DESIGN OF ELEVATED TANKS (REFER TO SEC. 4 OF ANSI/AWWA D100)

Sec. A.4.4 Cross-Braced, Multicolumn Elevated Tanks

A.4.4.1 *Steel riser*. Consideration should be given to specifying a corrosion allowance for load-bearing steel risers less than 36 in. (910 mm) in diameter or larger risers where accessibility may be difficult for inspection and painting.

Sec. A.4.5 Tank Plates

A.4.5.2 *Maximum unit stress*. To understand the difference between plates subject to a complete stress analysis and those not susceptible to a complete stress analysis, consider the following example. According to this standard, in an elevated tank having a vertical cylindrical shell supported by four columns attached to the shell and bottom, and having a suspended bottom with a central riser and a cone roof uniformly supported by the tank shell, the stresses in the ring of the cylindrical shell and the bottom to which the columns are attached cannot be accurately determined. The stresses in the roof and remainder of the shell can be completely determined.

A.4.5.3 *Cross-braced multicolumn tanks*. It is recognized that no specification for the design of elevated tanks can be sufficiently specific and complete to eliminate the necessity of judgment on the part of the designer. It is also recognized that strain-gauge surveys are a proper source of design information.

SECTION A.5: ACCESSORIES FOR ELEVATED TANKS (REFER TO SEC. 5 OF ANSI/AWWA D100)

Sec. A.5.1 Steel Riser

A riser larger than the 36-in. (910-mm) diameter may be required if the steel riser supports considerable load from the tank contents.

Consideration should be given to increasing the riser diameter of 36 in. (910 mm) in cold climates where the riser may freeze, unless other precautions are taken. The proper diameter will depend on the extent of the tank's use and the temperature of the water supplied. It may be necessary to insulate the riser and add a supplemental heat source to prevent freezing. See Chapter 10 of AWWA Manual M42, *Steel Water-Storage Tanks*.

This standard assumes that there is sufficient water replacement and circulation to prevent freezing in the tank and riser pipe. Where low usage may result in the possibility of freezing, water should be replaced or heated to prevent freezing. See NFPA* 22 for heater sizing. Where reference to ice damage is discussed in the standard, it is in anticipation of improper operation rather than an implied approval of an icing condition.

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

Sec. A.5.3 Overflow

An overflow protects the tank from overpressure, overload, and possible catastrophic failure if 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.5.4 Access

A.5.4.1 *General.* In November 2016, OSHA issued a new rule updating General Industry Walking-Working Surfaces and Fall Protection standards. Effective in November 2018, cages are no longer considered a ladder safety system or personal fall arrest system.

Consideration should be given to inspection personnel and maintenance personnel when specifying equipment in excess of OSHA minimum requirements.

A.5.4.2.5 Roof ladder. Regardless of the access protection provided to roof hatches and vents, weather conditions on tank roofs are extremely variable, and workers are expected to exercise good judgment and follow applicable regulations in matters of safety.

A.5.4.2.6 Inside tank ladder. Inside tank ladders are not recommended for cold climates where ice may form.

Sec. A.5.5 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.5.6 Antennas and Related Equipment

A.5.6.1 *General.* The following recommendations may be applied to new and existing tanks. A tank may be host to several wireless communication carriers. For new tanks, provisions for antennas and related equipment should be considered at the design stage. However, it should be noted that every tank site is not suitably located for a wireless communication installation and should be investigated for suitability.

Before antennas and related equipment are installed on existing tanks, the condition of the tank and support structure should be determined and necessary repairs performed.

Loads from antennas and related equipment should be distributed to prevent distress or buckling of the tank or support structure. Reinforcing plates, stiffeners, or both should be used to spread the loads. The ability of the support structure, anchorage, and foundation to withstand loads of this standard plus loads from antennas and related equipment should be verified.

Epoxy and stud-welded attachments should only be used for minor cable supports on horizontal surfaces. Any damage to existing coating should be repaired, and attachments should be seal welded or caulked.

Antenna cables should be arranged and located to allow adequate protection during abrasive blasting and recoating operations. Neatly arranged cables offset from the tank or support structure will allow maintenance without removing the cables.

The antenna installation must comply with FAA requirements. If new antennas are installed on the top of the tank, FAA obstruction lighting may need to be installed or raised to meet FAA guidelines.

Bolted components used on the exterior of the tank or support structure should be painted, seal welded, or caulked to prevent corrosion and rust streaking.

When electrical components are installed inside the support structure of single pedestal tanks, the effects of high humidity and condensation should be considered and the equipment adequately protected or waterproofed.

Interference with interior tank components should be avoided.

Clamps or bands used to attach equipment and cables to columns of multicolumn tanks should be sealed or caulked to prevent corrosion and rust streaking. Such clamps and bands should be removable for maintenance and inspection.

Tubular columns of multicolumn tanks that are hermetically sealed should not be breached (i.e., punctured) because moisture will accumulate inside the leg and may eventually cause structural damage.

A.5.6.2 *Health and safety*. Precautions concerning radio frequency exposure of personnel should be considered. Precautions should be taken to prevent contamination of the stored water during the installation and maintenance of antenna and related equipment. Access to the tank interior should not be permitted during installation and maintenance of antennas and related equipment unless proper disinfection and safety procedures are followed.

The coating system on existing tanks should be checked for hazardous metals such as lead. Where hazardous metals are found, the environment, stored water, and workers should be protected from contamination during installation and maintenance of antennas and related equipment.

Access to antennas and related equipment should comply with Occupational Health and Safety Administration (OSHA) regulations. This may require a safety rail around the installation, anchor points on the tank roof for personnel tie-off, ladders, or other fall-prevention devices.

Antenna cables should be supported at regular intervals (about 4 ft [1.2 m] on center) in exposed locations. Antennas and related equipment should not interfere with OSHA-defined access. For instance, cables should not be attached to ladders or obstruct manholes and platforms. Cable ladders or other commercially available cable support systems should be used.

Where space is limited, such as in small-diameter access tubes, cables should be fitted to the access tube wall to maximize clearance.

Consideration should be given to providing additional paint scaffold supports if the antenna installation renders the existing system unusable.

A.5.6.3 *General workmanship.* Holes should not be cut in the tank and support structure without making provisions to support the loads acting at the cutout. The tank should be empty when penetrations are cut in the tank or support structure. If this is not possible, it is recommended that the water level be lowered to 50 percent of maximum capacity and stiffeners added to strengthen the tank or support structure before openings larger than 3 in. (76 mm) in diameter are cut.

Stiffeners act to prevent buckling and add area if the effective area of the fitting is not sufficient. For multiple penetrations, holes should be cut one at a time and fittings welded in place before the next hole is cut. A clearance of 6 in. (152 mm) should be maintained between adjacent penetrations or stiffeners for adequate welder access.

A.5.6.4 *Welding.* The following welding recommendations should be considered.

Welding should be in accordance with ANSI/AWWA D100 Sec. 8, Welding; Sec. 10, Erection; and Sec. 11, Inspection and Testing.

Welds should be made with E7018 electrodes or other low hydrogen welding process and should be free of burrs and undercuts. Welds should meet the requirements of ANSI/AWWA D100, Sec. 11.

No welding should be performed when the ambient temperature is below 32°F (0°C) unless the cold weather welding requirements of ANSI/AWWA D100, Sec. 10 are followed.

Welded attachments to exterior surfaces and interior surfaces exposed to condensation should be seal welded to prevent rust streaking.

Penetrations should not intersect weld seams. Penetrations should clear existing weld seams by at least 6 in. (152 mm). If this clearance is not possible, an inspection should be made of adjacent weld seams that may be affected by local welding.

Welding to the tank or access tube should not be performed with water directly opposite the weld. The water level should be lowered at least 2 ft (0.6 m) below the point of welding to avoid welding problems.

Welding on the exterior may damage the interior coating opposite the weld. Damage to the interior coating system should be repaired during installation or when the tank is taken out of service. Damage to the exterior coating system should be repaired after completion of the antenna installation and should be compatible with the existing coating system. It is recommended that a one-year anniversary inspection be made to evaluate the performance of the coating repairs. ANSI/AWWA D102 should be consulted for coating requirements.

Galvanized components should not be welded directly to the tank or support structure. Galvanized surfaces must be ground free of galvanizing before welding.

Sec. A.5.7 Galvanic Corrosion

For tank components exposed to the stored water, construction with dissimilar metals more noble (stainless steel, copper, brass) 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 maximum water level (MWL) on open top tanks can cause dissimilar metals corrosion of the carbon steel tank component(s) to which they are attached.

In most cases, stainless steel 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 details that can provide isolation.

In cases where electrical isolation of stainless-steel internal components below the MWL has been determined to not be possible, then it is recommended that the stainless-steel components be lined and coated with a suitable dielectric coating. It is the responsibility of the owner or specification writer and not the painting contractor to determine whether any dissimilar metals exist and, if so, how the dissimilar metals are to be treated with respect to the coating system.

It should be noted that the solutions to dissimilar metals corrosion generally are contained in the details and configuration of the construction of the tank and its components. The coating specification cannot change those details, but it can address how and where to apply dielectric coatings to reduce the exposed cathodic surface, thereby reducing the effects of dissimilar metals corrosion.

SECTION A.7: ACCESSORIES FOR GROUND-SUPPORTED STANDPIPES AND RESERVOIRS (REFER TO SEC. 7 OF ANSI/AWWA D100.)

Sec. A.7.2 Pipe Connections

Thin shells do not possess inherent strength to resist out-of-plane loads or out-of-plane moments imposed by attached piping connections and, as such, are susceptible to tearing of the shell under substantial loads of this nature. Accordingly, ANSI/AWWA D100, ASCE 7, IBC, and other state and local codes prohibit piping configurations that impose significant loads on the tank. This requirement applies to all load combinations that may impose significant loads on the tank. Loads may be induced as a result of settlement, uplift, or movement of pipe due to line pressure. Use caution when designing attached piping systems that incorporate mechanical devices intended to provide piping flexibility. It is important to understand how much force is required to mobilize the flexibility of the device. Some devices intended to provide piping flexibility will resist movement and impart substantial loads before the device flexibility is activated. For such devices, special detailing of the piping, fittings, supports, and foundations is required to ensure that significant mobilizing forces are not transferred to the tank. Design for piping flexibility must accommodate potential seismic displacement and uplift of the tank, movement of the piping system, differential settlements, pressure-induced movement of piping components (valves open or closed), and the forces generated before mobilizing joint flexibility.

Acceptable details for shell penetrations can be found in API 650.

Sec. A.7.3 Overflow

An overflow protects the tank from overpressure, overload, and possible catastrophic failure if 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 Access

A.7.4.1 *General.* In November 2016, OSHA issued a new rule updating General Industry Walking-Working Surfaces and Fall Protection. Effective November 2018, cages are no longer considered a ladder safety system or personal fall arrest system. Consideration should be given to inspection personnel and maintenance personnel when specifying equipment in excess of OSHA minimum requirements.

A.7.4.2.3 Roof ladder. Regardless of the access protection provided to roof hatches and vents, weather conditions on tank roofs are extremely variable, and workers are expected to exercise good judgment and follow applicable regulations in matters of safety.

A.7.4.2.4 Inside tank ladder. Inside tank ladders are not recommended for cold climates where ice may form.

A.7.4.3 *Roof openings*. Additional roof openings may be required for ventilation during painting.

A.7.4.4 *Shell manholes*. Additional shell manholes may be required for ventilation during painting.

Sec. A.7.5 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.7.7 Galvanic Corrosion

For tank components exposed to the stored water, construction with dissimilar metals more noble (stainless steel, copper, brass) 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 maximum water level (MWL) on open top tanks can cause dissimilar metals corrosion of the carbon steel tank component(s) to which they are attached.

In most cases, stainless steel 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 details that can provide isolation.

In cases where electrical isolation of stainless-steel internal components below the MWL has been determined to not be possible, then it is recommended that the stainless-steel components be lined and coated with a suitable dielectric coating. It is the responsibility of the owner or specification writer and not the painting contractor to determine whether any dissimilar metals exist and, if so, how the dissimilar metals are to be treated with respect to the coating system.

It should be noted that the solutions to dissimilar metals corrosion generally are contained in the details and configuration of the construction of the tank and its components. The coating specification cannot change those details, but it can address how and where to apply dielectric coatings to reduce the exposed cathodic surface, thereby reducing the effects of dissimilar metals corrosion.

SECTION A.8: WELDING (REFER TO SEC. 8 OF ANSI/AWWA D100.)

Sec. A.8.2 Qualification of Welding Procedures, Welders, and Welding Operators

A.8.2.1 *Qualification of welding procedure specifications.* All welding is required to be performed in accordance with welding procedure specifications (WPSs) that are developed by the employer following applicable rules of ASME BPVC Sec. IX, AWS D1.1/D1.1M, or AWS B2.1/B2.1M.

AWS B2.1/B2.1M standard welding procedure specifications (SWPSs) are acceptable when they meet the requirements of ASME BPVC Sec. IX, Article V. This requires welding and testing a test plate with one groove weld in accordance with clause QW-510(d) before an SWPS is permitted to be used.

A.8.2.1.1 Partial joint penetration welds. Partial joint penetration welding procedures may be qualified with complete joint penetration welding procedure specifications. However, the qualification of the complete joint penetration procedure does not include provisions for achieving a partial strength joint. Therefore, qualification of partial joint penetration welds requires an additional reduced-section tension test to demonstrate that the specific joint details will be capable of producing a welded joint with strength at least equivalent to two-thirds that of the tested base metal. A separate qualifying test is required whenever any of the following changes in joint details occurs: change in groove type, decrease in groove angle, decrease in root opening, or an increase in the root face percentage where root face percentage is defined as the dimension of the root face divided by the thickness of the thinner plate in the joint. Alternatively, complete joint penetration welds may be used.

A.8.2.1.4 Acceptable welding procedure specifications. Any method that correlates all tank joints with welding procedures that are acceptable for each joint is acceptable. For example, construction drawings may indicate which welding procedures may be used on each joint by reference, by standard joint details, or by noting welding procedure specifications designations in the tail of the welding symbol or with the joint detail. Alternatively, each welding procedure specification could indicate for which tank joints the procedure is applicable.

A.8.2.1.5 Providing welding procedure specifications. Certified copies of welding procedure specification and procedure qualification records may be specified to supplement the information provided in the written inspection report (see Sec. 11.2).

A.8.2.2 *Qualification of welders and welding operators*. Welder qualification testing is not portable from one employer (constructor or manufacturer) to another. Welder qualification testing is based on manufacturer-specific welding procedure specifications.

A.8.2.2.1 Test records. The records of welder testing and welder identification are certified for authenticity and validity of the records. These records are part of the written inspection report (see Sec. 11.2).

Sec. A.8.3 Weld Joint Records

The record of welders employed on each joint is part of the written inspection report (see Sec. 11.2).

Sec. A.8.9 Tubular Column and Strut Sections

The interior of multiple-leg tank columns less than 30 in. (750 mm) in diameter is considered inaccessible for two-sided welding.

Sec. A.8.13 Lap Restrictions for Welded Lap Joints

The restriction on maximum lap of roof plates only applies if none of the specified means are provided to support the unwelded plate edge.

Sec. A.8.14 Minimum Size of Fillet and Seal Welds

A.8.14.1.1 Adjustment for root opening. The size of fillet welds must be increased by the amount of the root opening in excess of $\frac{1}{16}$ in. (1.6 mm) to maintain the strength of the weld. The maximum root opening of $\frac{3}{16}$ in. (4.76 mm) is specified to maintain the joint configuration within good welding practice.

Sec. A.8.17 Corrosion Protection

Welded joints in condensing surfaces and exterior welded joints not shielded from rain or rain runoff should be seal welded or caulked.

Interior joints not in direct contact with stored water and not part of a condensing surface do not require protection (i.e., no seal weld or caulk).

This standard does not require seal welding of exterior joints that are not exposed to rain or rain runoff (e.g., protected shell girder-to-shell welds). The likelihood of exposure to wind-driven moisture should be considered when deciding if such joints should be seal welded.

SECTION A.9: SHOP FABRICATION (REFER TO SEC. 9 OF ANSI/AWWA D100.)

Sec. A.9.4 Rolling

For ground-supported tanks, Table 15 may be used as a guide for determining if rolling is recommended.

Sec. A.9.5 Double-Curved Plates

Double-curved tank bottoms, shells, and roofs may be assembled in the shop, if necessary, to ensure that they will fit properly in the field.

SECTION A.10: ERECTION (REFER TO SEC. 10 OF ANSI/AWWA D100.)

Sec. A.10.1 Welds

A.10.1.1 *Weather and temperature conditions.* It is recommended that no welding be done when the base metal temperature falls below 0°F (–18°C). Refer to Sec. 10.3.2 and Sec. 10.4.2 for requirements when welding below 0°F (–18°C).

Sec. A.10.4 Low-Hydrogen Electrodes and Welding Processes

A.10.4.2 *Low temperatures.* The use of low-hydrogen electrodes will be helpful when welding is performed at low temperatures, especially in the welding of materials more than 1 in. (25 mm) thick.

Sec. A.10.6 Tank Assembly

Construction openings are used by tank constructors to facilitate access into the tank for numerous construction purposes, including the placement, use, and removal of construction, welding, or painting equipment, as well as for ease of personnel access, ventilation, lighting, and safety considerations. The closure of all construction openings must comply with all design, fabrication, welding, inspection, and tolerance requirements of ANSI/AWWA D100. ANSI/AWWA D100 does not contain special requirements for the use, design, closure, or inspection of construction openings. Unless construction openings encompass the entire height of a shell course, it is recommended that the openings be round or constructed with rounded corners. When openings encompassing the entire height of a shell course are used, consideration should be given to cutback of the horizontal seams beyond the vertical sides of openings. Lap-welded shell closure plates are not recommended unless the details of the construction opening preclude post-installation of a butt-welded closure plate. Sec. 9.2 of API 653 can be consulted for guidelines on butt-welded closure plates. Judgment may be required in the application of API 653 for use on water tanks because it is written for use on petroleum oil storage tanks designed in accordance with API 650.

A.10.6.6.1 Local deviation from theoretical shape. It may be necessary to evaluate gauge lengths other than the specified length, $L_x = 4\sqrt{Rt}$, when appropriate for the method of construction. A gauge length of $2\sqrt{Rt}$ is recommended at the base of the shell. An imperfection ratio (e_x/L_x) of 0.001 shall be maintained when evaluating various gauge lengths.

SECTION A.11: INSPECTION AND TESTING (REFER TO SEC. 11 OF ANSI/AWWA D100.)

Sec. A.11.2 Inspection Report

It is recommended that the inspection of the radiographs be made immediately after the first vertical joints are welded to detect unacceptable welding before extensive welding is completed.

When field inspection by a qualified inspector is provided, the inspector should do the following:

1. Examine the credentials of the welders and witness the operators' qualification tests, if such tests are required.
2. Examine all radiographs and make a written report stating whether such radiographs are acceptable and, if not, reasons they are not and whether in the inspector's opinion the welding covered by such test specimens is of the quality required by this standard and is in accordance with good workmanship.
3. If the first welds by any operator are unsatisfactory, the inspector should require additional tests of welds by that operator. If such tests continue to be unsatisfactory, the welder should be prohibited from doing further welding.
4. After the initial welding has been satisfactorily completed, the inspector may leave the job and return only at such times as, in his or her judgment, it is necessary to confirm the quality of the welding on the remaining seams.

Sec. A.11.4 Inspection of Welded Joints

A.11.4.1.2 (4) Additional inspection should be considered for splice welds in rim angles subject to significant primary stress caused by horizontal thrust from self-supporting roofs.

A.11.4.2 *Visual inspection*. Additional information related to visual inspection can be found in AWS D1.1/D1.1M, Sec. 5.24 and Figure 5.4, and in AWS A3.0M/A3.0, Figure 32.

A.11.4.2.1 Maximum permissible undercut. Undercut not exceeding the limits of Sec. 11.4.2.1 may occur at each location where the weld surface and base metal surface meet as shown in Figure A.2.

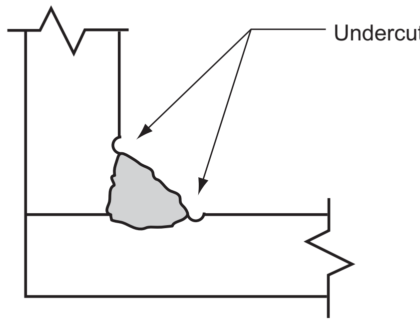


Figure A.2 Typical undercut

A.11.4.4.1 Circumferential butt joints. Tubular support columns for elevated tanks are usually controlled by compression. Columns subjected to large uplift forces from wind or seismic loads should also be checked to ensure that the allowable tensile stress reduced by the appropriate joint efficiency is not exceeded.

Sec. A.11.10 Testing

A.11.10.1.2 Air-pressure or vacuum testing. 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. About 30 in. (760 mm) of seam under test 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 porosity in the seam is indicated by bubbles or foam produced by air sucked through the welded 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).

SECTION A.12: FOUNDATION DESIGN (REFER TO SEC. 12 OF ANSI/AWWA D100.)

Sec. A.12.1 General Requirements

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

The proper design and installation of foundations for standpipes and reservoirs are important to ensure uniform and minimum settlement.

Sec. A.12.6 Foundations for Ground-Supported Flat-Bottom Tanks

The language has been revised to clearly indicate that the oiled sand base is the recommended default cushion for ground-supported flat-bottom tanks on concrete ringwalls. Type 3 and 5 foundations are recommended only for desert climates unless provisions made for drainage from inside the retainer ring can ensure that water will not be trapped under the tank bottom or around the tank shell if the tank settles relative to the retainer ring.

The addition of hydrated lime to clean sand can be used to enhance under bottom corrosion protection.

The asphalt cutback oils used in oiled sand mixtures are the same types of oils used in construction of roads and road beds. The selection of the viscosity and curing characteristics of the oil are at the discretion of the contractor to suit local availability, climate conditions of the work site, and the contractor's installation methods. Commonly used oils include MC (medium curing) 70 and SC (slow curing) 250. Food-grade oils such as vegetable oil are not considered acceptable alternatives to the asphalt cutback oils due to quick degradation of that material.

Cathodic protection systems are available for under bottom corrosion protection.

Concrete surfaces to be grouted shall be free of oil, grease, laitance, and other contaminants. Concrete must be clean, sound, and roughened to ensure good bond. Prior to placement, concrete surface should be brought to a saturated surface-dry condition.

Sec. A.12.7 Detail Design of Foundations

A.12.7.1 *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.13: SEISMIC DESIGN OF WATER STORAGE TANKS (REFER TO SEC. 13 OF ANSI/AWWA D100.)

Sec. A.13.1 General

The required seismic design loads of this standard are based on the requirements of ASCE 7-16. Maps previously published in ANSI/AWWA D100 are now included by reference only to ASCE 7.

A.13.1.1 *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 used:

1. A response spectrum complying with the regulatory requirements may be used, provided that 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 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, then the following substitutions shall apply:

$$S_S = 2.5S_P \quad \text{(A.13-1)}$$

$$S_1 = 1.25S_P \quad \text{(A.13-2)}$$

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.13.2 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. 13.2.6 or the site-specific procedure of Sec. 13.2.7. The general procedure in which spectral response acceleration parameters for the MCE_R ground motions are derived using ASCE 7 Figures 22-1 through 22-6, 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. 13.2.7 is permitted to be used for any structure and shall be used when specified or required by the standard.

A.13.2.1 *Seismic Importance Factor, I_E.* Tanks are assigned a seismic importance factor based on the risk category determined by definitions in Sec. 3.

A.13.2.2 *Mapped acceleration parameters*. The maps for determination of acceleration parameters are included in ANSI/AWWA D100 by reference only to ASCE 7. Values for the acceleration parameters may also be obtained from web-based services such as the ASCE 7 Hazard Tool and the ATC Hazards by Location tool.

A.13.2.3 *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 uses this ASCE 7 approach. The site classification procedure is included by reference only to ASCE 7.

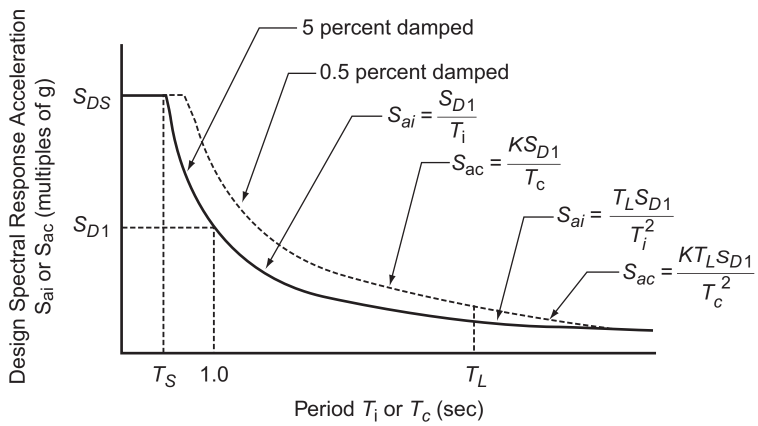


Figure A.3 Design response spectra—general procedure

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.

A.13.2.6.3 Design response spectra. The design response spectra for the general procedure (Eqs 13-5 through 13-9) are shown in Figure A.3.

A.13.2.7 *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. 13.2.6 and is required for tanks located on Site Class F soils.

A.13.2.7.2 Design response spectrum. Special care must be exercised when generating a design response spectrum from a site-specific spectrum with humps and jagged variations. ASCE 7 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 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 2.0-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. 13.2.6. The resulting site-specific design spectrum should be generated in accordance with Sec. 13.2.6.3.1 and should be smoothed to eliminate extreme humps and jagged variations.

A.13.2.7.2.1 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:

$$S_{ai,slide} = \tan 30^\circ \left(\frac{W_T}{W_i} \right) \quad \text{(A.13-3)}$$

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 13-23

W_i = weight of effective mass of tank contents that moves in unison with the tank shell, in pounds, determined by Eq 13-20 or 13-21

The sliding resistance in the above equation is based on a static coefficient of friction equal to tan 30°. The coefficient tan 30° is a best-estimate of coefficient of friction value for bottom plates placed on concrete, cane-fiber joint filler on concrete, or cushions constructed of oiled sand, sand, well-graded crushed stone or gravel, hydrated lime-sand mix, or asphaltic road mix. For allowable stress-based design, the maximum friction value that can be used for sliding resistance is equal to λ_E times the tan 30° coefficient (0.70 × tan 30° = 0.4). A plot of the above equation is shown in Figure A.4.

Sec. A.13.3 Cross-Braced, Column-Supported Elevated Tanks

Cross-braced, column-supported elevated tanks have performed favorably under seismic loads when details are such that yielding can develop in the bracing before failure of the connection or buckling of a strut.

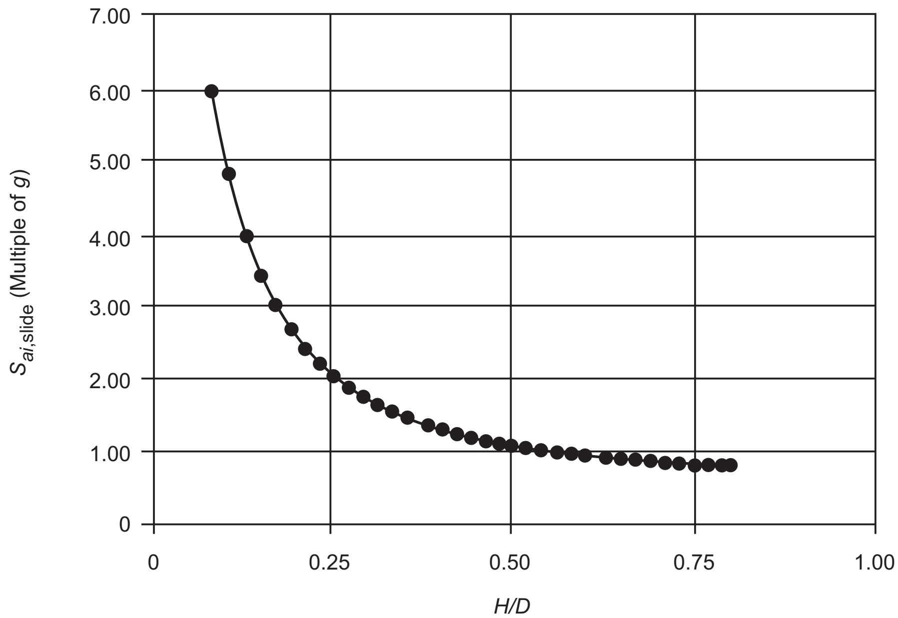


Figure A.4 Design spectral response acceleration that causes the tank to slide, $S_{ai,\,slide}$

A.13.3.3.6 Vertical design acceleration. Two methods of combining load effects from vertical and horizontal design acceleration are allowed. These are not explicitly contained within the standard for elevated tanks, but one example of the application of the SRSS method is displayed in the numerator of Eq 13-38a. An example of the direct sum method is displayed in the numerator of Eq 13-38b.

Sec. A.13.4 Pedestal-Type Elevated Tanks

Few pedestal-type elevated tanks have been subjected to large seismic forces. Those that have experienced seismic loading have survived with little or no damage. Because these structures are designed in the buckling range and are not capable of yielding to relieve load, they are susceptible to buckling under seismic overload and must be designed more conservatively. Structures in this category include tubular shaft and bent-plate pedestals. A compensating factor in design for slender pedestal tanks is that the natural periods are relatively long, so seismic load may be less than for a rigid structure.

A.13.4.3.3 Vertical design acceleration. Two methods of combining load effects from vertical and horizontal design acceleration are allowed. These are not explicitly contained within the standard for elevated tanks, but one example of the application of the SRSS method is displayed in the numerator of Eq 13-38a. An example of the direct sum method is displayed in the numerator of Eq 13-38b.

Sec. A.13.5 Ground-Supported Flat-Bottom Tanks

A.13.5.1 *Natural periods.* For the site-specific procedure, the natural period of the shell-fluid system may be determined using API 650 or reference 14, 16, or 17 of Sec. A.1.4.

The first mode sloshing wave period, T_c, may be determined by Eq 13-18, or by the graphical procedure using Eq A.13-4 and Figure A.5.

$$T_c = K_p \sqrt{D} \tag{A.13-4}$$

Where:

T_c = first mode sloshing wave period, in seconds
K_p = factor from Figure A.5 for the ratio of D/H
D = tank diameter, in feet

A.13.5.2 *Design overturning moment at the bottom of the shell.* The standard provides an equation (Eq 13-19) 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.

A.13.5.2.2 Effective weight of tank contents. The effective impulsive and convective weights W_i and W_c may be determined by Eqs 13-20 through 13-22, or by multiplying W_T by the ratios W_i/W_T and W_c/W_T obtained from Figure A.6. 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 Eqs 13-24 through 13-26, or by multiplying H by the ratios X_i/H and X_c/H obtained from Figure A.7.

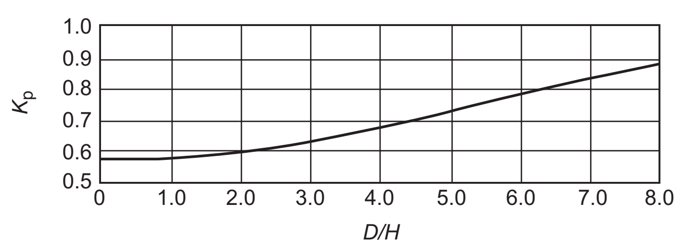


Figure A.5 Curve for obtaining factor K_p for the ratio D/H

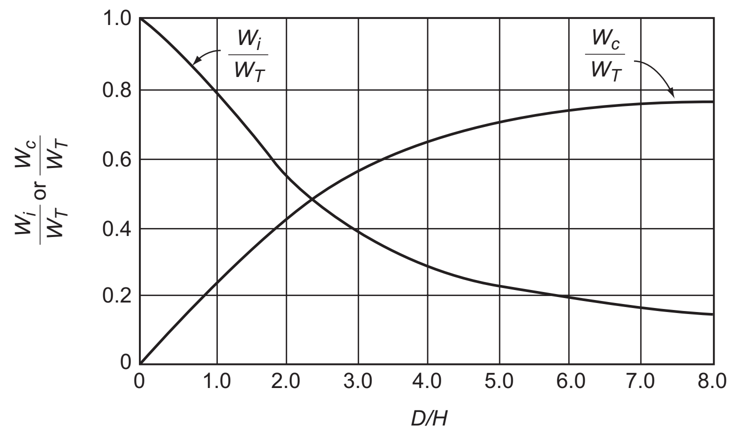


Figure A.6 Curves for obtaining factors W_i/W_T and W_c/W_T for the ratio D/H

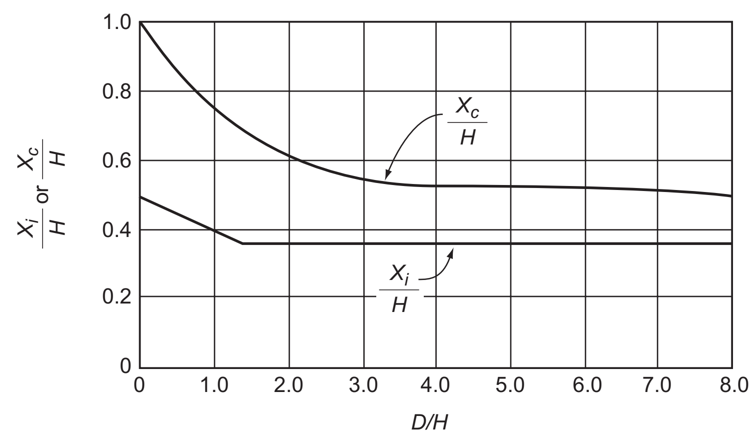


Figure A.7 Curves for obtaining factors X_i/H and X_c/H for the ratio D/H

A.13.5.3 *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 13-19). 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 13-28) 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 Eqs 13-29 through 13-31.

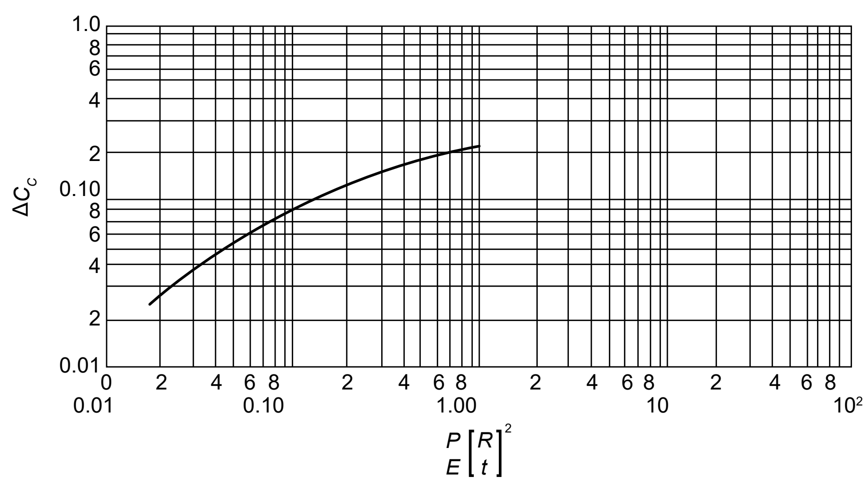
A.13.5.4.2.1 Longitudinal shell compression for self-anchored tanks. Longitudinal shell compression stress should be checked at the bottom of all shell courses.

The overturning moment at the bottom of each shell course may be calculated assuming that the overturning moment varies from zero at the top of the shell to M_s at the bottom of the shell, unless a more rigorous analysis is performed.

A.13.5.4.2.2 Longitudinal shell compression for mechanically anchored tanks. Longitudinal shell compression stress should be checked at the bottom of all shell courses.

The overturning moment at the bottom of each shell course may be calculated assuming that the overturning moment varies from zero at the top of the shell to M_s at the bottom of the shell, unless a more rigorous analysis is performed.

A.13.5.4.2.4 Allowable shell stress. The pressure-stabilizing buckling coefficient, ΔC_c, used to determine the seismic allowable longitudinal shell compression stress may be determined by Eqs 13-46 and 13-47 or obtained from Figure A.8.



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 A.8 Pressure-stabilizing buckling coefficient, ΔC_c, for self-anchored tanks

A.13.5.4.3 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. The SRSS method is displayed in the numerator of Eq 13-38a. The direct sum method is displayed in the numerator of Eq 13-38b.

A.13.5.4.6 Sliding check. When a sliding check is specified and the design shear at the top of the foundation exceeds the allowable lateral shear V_{ALLOW}, the tank must be reconfigured, additional shear resistance must be provided, or both.

Sec. A.13.6 Freeboard

If sufficient freeboard for the sloshing wave is not provided, some loss of contents and roof damage may occur if the tank is completely full during an earthquake. When less than design freeboard is provided, the roof and its supporting structure must be designed to resist the hydrostatic pressure equal to the design freeboard minus the freeboard provided. The portion of the convective mass considered to be confined by the roof shall be considered as impulsive mass.

Sec. A.13.7 Piping Connections

Thin shells do not possess inherent strength to resist out-of-plane loads or out-of-plane moments imposed by attached piping connections and, as such, are susceptible to tearing of the shell under substantial loads of this nature. Accordingly, ANSI/AWWA D100, ASCE 7, IBC, and other state and local codes prohibit piping configurations that impose significant loads on the tank. This requirement applies to all load combinations that may impose significant loads on the tank. Loads may be induced as a result of settlement, uplift, or movement of pipe due to line pressure. Use caution when designing attached piping systems that incorporate mechanical devices intended to provide piping flexibility. It is important to understand how much force is required to mobilize the flexibility of the device. Some devices intended to provide piping flexibility will resist movement and impart substantial loads before the device flexibility is activated. For such devices, special detailing of the piping, fittings, supports, and foundations is required to ensure that significant mobilizing forces are not transferred to the tank. Design for piping flexibility must accommodate potential seismic displacement and uplift of the tank, movement of the piping system, differential settlements, pressure-induced movement of piping components (valves open or closed), and the forces generated prior to mobilizing joint flexibility.

A.13.7.1 *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

$$y_u = \frac{F_y L^2}{83{,}300 t_b} \tag{A.13-5}$$

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 13-34

t_b = design thickness of the bottom annulus, in inches

SECTION A.14: ALTERNATIVE DESIGN BASIS FOR STANDPIPES AND RESERVOIRS (REFER TO SEC. 14 OF ANSI/AWWA D100.)

Sec. A.14.1 Alternative Design Basis

Sec. 3 of ANSI/AWWA D100 provides rules for the design of cylindrical shell plates for standpipes and reservoirs based on a unit tensile stress of 15,000 psi (103.4 MPa) with an 85 percent joint efficiency producing a net allowable tensile working stress of 12,750 psi (87.9 MPa). Even though some of the steels specified for use for shell plates may have ductile to brittle transition ranges greater than the ambient temperature of use, the working stress under Sec. 3 designs has been sufficiently low that brittle fractures have not occurred and the Sec. 3 design philosophy has resulted in safe structures.

Steels with controlled chemical and alloy compositions that justify higher tensile working stresses are available. Sec. 14 provides an alternative design basis to that specified in Sec. 3 for shell plates for welded steel standpipes and reservoirs. Included are special design rules, temperature limits, working stress levels, restrictions on shell penetrations, and additional inspection requirements when these steels are used with the higher working stress design. The higher working stresses allowed under Sec. 14 are comparable to working stress levels allowed in other steel construction industries (allowable stress design provisions of ANSI/AISC 360).

Steels with improved notch toughness are specified, more rigid requirements pertaining to shell penetrations are incorporated, and increased material requirements, weld inspection, and testing are required.

For tanks located in highly active seismic regions, consideration should be given to the reduced seismic buckling resistance that Sec. 14 tanks will have compared with the same size tank built to Sec. 3 design stresses. Because the Sec. 3 tank will be thicker than its Sec. 14 counterpart, the Sec. 3 tank has inherently higher resistance to these loads. Selection of proper seismic design level is equally important to both Sec. 3 and Sec. 14 tanks. Generally, Sec. 14 designs are suitable for tanks over 1,000,000-gal (3,785.4-m^3) nominal capacity.

A.14.1.1 *Applicability*. When Sec. 14 is used, the requirements of Sec. 2.2.3.2 are superseded in addition to those of Sec. 3. Other requirements of the standard still apply, except where superseded by specific provisions of Sec. 14.

A.14.1.4 *Welding procedure qualification*. When Sec. 14 design is used, notch toughness requirements may apply to the shell material, electrodes, and welding procedures.

Sec. A.14.2 Materials

A.14.2.4 *Design metal temperature*. Normally, the design metal temperature for a tank is determined in accordance with Sec. 14.2.4. Special conditions that might support the use of an alternate DMT are heated or insulated tanks.

A.14.2.6 *Other material requirements*. Decorative pilasters are not a necessary tank component and require particular care when welded directly to higher stressed shells.

A.14.2.6.1 Plate necks, reinforcing plates, and insert plates. Typical details for attaching necks, reinforcing plates, and insert-type reinforcements for nozzles and manholes, except for ASTM A517/A517M material, may be found in API 650.

A.14.2.6.3 (6) Temporary attachments to shell courses of any Sec. 14 materials should be made prior to welding of the shell joints.

Sec. A.14.3 General Design

The increased stress level permitted under the provisions of this section will result in comparatively thin shells, and wind girders intermediate between roof and bottom may be required. See Sec. 3.5 for formulas used to determine whether shell stiffeners are required.

No allowance for corrosion is required by this section. It is assumed that suitable coatings or other protection will be maintained so that corrosion does not occur. If an allowance for corrosion is desired, the allowance for parts that will be in contact with water and for parts that will not be in contact with water (see Sec. 3.9) shall be specified.

A.14.3.2.2 Analysis. Short tanks will require thicker shells and tall tanks will require thinner shells using the formula in Sec. 3.7 when compared to the shell analysis theory.

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APPENDIX B

Default Checklist

This appendix is for information only and is not a part of ANSI/AWWA D100.

The following default checklist is intended to assist the user when specifying a tank using this standard. The standard contains many options. The following checklist summarizes the options and the corresponding default for each option if no further direction is provided.

Section	Option	Default
1.2	Bottom capacity level (BCL) for ground-supported flat-bottom tanks	The BCL shall be the water level in the tank when the tank is emptied through the specified discharge fittings.
1.2	Maximum operating level (MOL)	The MOL shall be taken as the MWL.
1.3	Details of welded joints	Furnishing details of welded joints is not required.
2.1.1	Mill test reports	Furnishing mill test reports is not required.
2.2.11	Steel pipe joints	Steel pipe joints may be screwed, flanged, or welded.
3.1.1	Risk category	Risk Category IV shall be used.
3.1.4.1	Ground snow load for areas designated CS or for sites at elevations above the limits indicated in ASCE 7, Figure 7-1.	The ground snow load for the general area, shown in ASCE 7, Figure 7-1, will be used for these areas.
3.1.6.1	Basic wind speed for special wind regions	The basic wind speed for the general area, shown in ASCE 7, Figure 26.5, will be used for special wind regions.
3.1.6.2	Wind exposure category	Exposure C
3.1.6.3	Topographic effects	Wind loads shall be based on a topographic factor, K_{zt}, of 1.0.
3.1.6.7.2	Projected area for wind loads on shrouds	The projected area of the shroud shall be 6 ft (1.8 m) greater than the height of the structure and 6 ft (1.8 m) wider than the projected area of the structure.
3.3.3	Snow load combined with wind or seismic loads	Combining snow load with wind or seismic loads is not required.
3.5.3	Shell thickness for intermediate shell girders	The average, as-ordered shell thicknesses shall be used to determine height, *h*, in Eq 3-33.

Section	Option	Default
3.6.1.6	Painting of contact surfaces between roof plates and rafters	Priming or painting of contract surfaces between roof plates and rafters is not required.
3.9.1	Corrosion allowance	Corrosion allowance is not required for parts in contact with water and parts not in contact with water.
3.9.1	General application of corrosion allowance	The specified corrosion allowance shall be added to the thickness required by design, and not the minimum thickness. See Sec. 3.9.3 for exception for bottom plates for ground-supported flat-bottom tanks.
3.9.2	Application of corrosion allowance for structural sections	The specified corrosion allowance shall be applied as a total per element (e.g., web or flange), and not per surface.
4.3	Head range for elevated tanks	A variation of ±2.5 ft (±0.76 m) in the head range is allowed.
4.4.1	Steel riser	Riser shall be wet.
4.4.3.1	Tightening of diagonal tension members	Diagonal tension members shall be furnished with turnbuckles for tightening. Heat shrinking or other prestressing devices may be used when available turnbuckle sizes are exceeded.
5.1	Steel riser diameter	The minimum riser diameter shall be 36 in. (910 mm) in localities where freezing temperatures occur.
5.1.1	Riser safety grill	A riser safety grill is not required.
5.2.1	Removable silt stop	A removable silt stop is not required.
5.3	Overflow configuration	The overflow may be internal or external.
5.3	Overflow outlet	The outlet of the overflow shall be covered with a coarse, corrosion-resistant screen equivalent to ⅜ in. (9.5 mm) or larger mesh, or a flap valve.
5.3	Overflow pipe material	The overflow pipe shall be steel pipe, with screwed or welded connection if less than 4 in. (102 mm) in diameter, or with flanged or welded connection if 4 in. (102 mm) in diameter or larger.
5.4.2.1	Skid-resistant ladder rungs	Skid-resistant ladder rungs are not required.
5.4.2.2	Tower ladder	For multicolumn tanks, a tower ladder shall provide access from a point 10 ft (3.0 m) above grade.
5.4.2.6	Inside tank ladder	An inside tank ladder is not required.
5.6	Loads from antennas and related equipment	Design of the tank, support structure, and foundation for loads from antennas and related equipment is not required.
7.2.1	Removable silt stop	A removable silt stop is not required.

Section	Option	Default
7.3	Overflow configuration	The overflow may be internal or external.
7.3	Overflow outlet	The outlet of the overflow shall be covered with a coarse, corrosion-resistant screen equivalent to ⅜ in. (9.5 mm) or larger mesh, or a flap valve.
7.3	Overflow pipe material	The overflow pipe shall be steel pipe, with screwed or welded connection if less than 4 in. (102 mm) in diameter, or with flanged or welded connection if 4 in. (102 mm) in diameter or larger.
7.4.2.1	Skid-resistant ladder rungs	Skid-resistant ladder rungs are not required.
7.4.2.2	Outside tank ladder	Outside tank ladder shall provide access from a point 10 ft (3.0 m) above the bottom of the tank bottom.
7.4.2.4	Inside tank ladder	An inside tank ladder is not required.
7.6	Loads from antennas and related equipment	Design of the tank and foundation for loads from antennas and related equipment is not required.
8.2.1.5	Welding procedure specifications and supporting procedure qualification records	Furnishing welding procedure specifications and supporting procedure qualification records is not required.
8.4.2	Butt joints subject to secondary stress for base metals of thickness greater than ⅜ in. (9.5 mm)	Joints shall be double-welded and may be partial or complete joint penetration welds.
8.14.2	Seal welding	Seal welding is not required.
9.2	Straightening	Heat may be used to straighten severe deformations.
10.3.1	Preheat	The base metal within a distance of four times the plate thickness from the location where welding is started shall be preheated to at least the preheat temperature specified in Sec. 10.3.2, and that temperature shall be maintained for a distance of four times the plate thickness ahead of the arc as welding progresses.
11.2	Written inspection report	A written report confirming that the work was inspected as set forth in the standard is not required.
11.2	Radiographs and inspection records	Furnishing radiographs and inspection records is not required.
11.10.3	Hydrotest	The tank shall be hydrotested after painting.
12.1.2	Design snow load	Combining snow load with wind or seismic loads is not required.

Section	Option	Default
12.4.7	Tolerances on anchor bolt installation for cross-braced multicolumn tanks	The design of anchor bolt and anchor bolt attachments shall accommodate, and installation shall comply with, the tolerances given in Sec. 12.4.7.
12.5.3	Tolerances on anchor bolt installation for single-pedestal tanks	The design of anchor bolt and anchor bolt attachments shall accommodate, and installation shall comply with, the tolerances given in Sec. 12.5.3.
12.6	Sand cushion grade	The sand cushion shall be graded to slope uniformly upward to the center of the tank with a minimum slope of 1 in. (25 mm) vertical to 10 ft (3.0 m) horizontal.
12.6	Sand cushion	An oiled sand cushion shall be provided.
12.6.1 (1)	Grout	Grout may be 1:1.5 cement-sand grout or commercial grout.
12.6.1 (2)	Grout	Grout may be 1:1.5 cement-sand grout or commercial grout.
12.6.3	Tolerances on anchor installation for ground-supported flat-bottom tanks	The design of anchors and anchor attachments shall accommodate, and installation shall comply with, the tolerances given in Sec. 12.6.3.
12.7.1	Height of foundation above ground	Top of foundation shall be at least 6 in. (152 mm) above finished grade.
12.7.4	Buoyancy	Design of the foundation for the effects of buoyancy is not required.
12.8	Concrete work	Concrete work shall comply with ACI 301.
12.8.1	Placing concrete	Concrete for the riser and column piers shall be placed monolithically.
13.1.1	Design earthquake ground motion	Design earthquake ground motion shall be determined by the general procedure (Sec. 13.2.6) or, if required by the standard, the site-specific procedure (Sec. 13.2.7).
13.2.3	Site Class	Site Class D shall be used when the soil properties are not known in sufficient detail to determine the site class, provided that Site Class E or F soils are not present at the site.
13.2.7.1	Site-specific procedure	The site-specific procedure must be used only if required by the standard.
13.2.8.1	Design spectral response acceleration for elevated tanks	The design spectral response acceleration shall be taken from a design spectrum determined by the general procedure (Sec.13.2.6) or, if required by the standard, the site-specific procedure (Sec.13.2.7).

Section	Option	Default
13.2.8.2	Design spectral response acceleration for ground-supported flat-bottom tanks	The design spectral response acceleration shall be taken from a design spectrum determined by the general procedure (Sec. 13.2.6) or, if required by the standard, the site-specific procedure (Sec. 13.2.7).
13.3.3.6	Vertical design acceleration for cross-braced, column-supported elevated tanks	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 the vertical design acceleration being multiplied by 0.40, or by the SRSS method.
13.4.3.3	Vertical design acceleration for pedestal-type elevated tanks	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 the vertical design acceleration being multiplied by 0.40, or by the SRSS method.
13.5.4.3	Vertical design acceleration for ground-supported flat-bottom tanks	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 the vertical design acceleration being multiplied by 0.40, or by the SRSS method.
13.5.4.5	Seismic design of roof framing and columns for ground-supported flat-bottom tanks	Seismic design of roof framing and columns is not required.
13.5.4.5	Seismic design of roof framing and columns for ground-supported flat-bottom tanks	If seismic design of roof framing and columns is specified, combining live load with seismic loads is not required.
13.5.4.6	Sliding check for ground-supported flat-bottom tanks	A sliding check for seismic loads is not required.
13.7.1	Piping flexibility	The piping system shall accommodate the design displacements given in Table 26.
14.1.1	Applicability of Sec. 14	The design basis shall be Sec. 3.
14.2.4	Design metal temperature	The design metal temperature shall be the lowest one-day mean ambient temperature from Figure 6 plus 15°F (–9°C).
14.4.5	Certified welding inspector (CWI) shall be responsible for required weld inspections	A certified welding inspector is not required.

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