ANSI/AWWA D100-11
(Revision of ANSI/AWWA D100-05)

American Water Works Association

The Authoritative Resource on Safe Water®

AWWA Standard

Welded Carbon Steel Tanks for Water Storage

STANDARD
Since 1881

Approved American National Standard
ANSI

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AWWA Standard

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Foreword

This foreword is for information only and is not a part of ANSI/AWWA 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/sec). 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 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, 550 N.W. LeJeune Road, Miami, FL 33126.
‡ ASME International, Three Park Avenue, New York, NY 10016.
§ American Concrete Institute, 38800 Country Club Dr., 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 Section II of the foreword, and the sections dealing with foundations were incorporated into Section 12. Section 11 was revised to include inspection and testing requirements that were formerly in Section 11 and Section 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, Section VIII, or to the identical charts in American Petroleum Institute (API)* Standard 650, Welded Steel Tanks for Oil Storage. A section covering permissible inspection by air carbon arc gouging was added to Section 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 workpoint 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 American Society for Testing and Materials (ASTM)† A36, Specification for Structural Steel, were allowed to be used, provided 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 5⁄16 in. (7.9 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 Section 13. In addition, a new section was added

* American Petroleum Institute, 1220 L Street, NW, Washington, DC 20005.
†ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428.

to Section 13 to permit scaling down to specific site response spectra when local seismic data are available.

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 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 Section 14, and reference standards were moved to Section 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 Section 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 (Method 2 and Method 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 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 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 1/16-in. (1.59-mm) additional shell thickness requirement for flush-type cleanouts was eliminated to match the current requirements of API 650. The requirement that inlet

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

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

protection be removable was added for elevated tanks. Electrical isolation requirements were added for dissimilar metals inside the tank below the TCL.

Section 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 4/3 times the published 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 Method 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.

* Building Seismic Safety Council, 1090 Vermont Avenue, N.W., Suite 700, Washington, DC 20005.

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.

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

The last edition was approved by the AWWA Board of Directors on June 12, 2005. This edition was approved on Jan. 23, 2011.

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 American Water Works Association Research Foundation (AwwaRF, now Water Research Foundation*) 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. An advisory program formerly administered by USEPA, Office of Drinking Water, discontinued on Apr. 7, 1990.
2. Specific policies of the state or local agency.
3. Two standards developed under the direction of NSF, NSF‡/ANSI 60, Drinking Water Treatment Chemicals—Health Effects, and NSF/ANSI 61, Drinking Water System Components—Health Effects.
4. Other references, including AWWA standards, *Food Chemicals Codex*, *Water Chemicals Codex*,§ and other standards considered appropriate by the state or local agency.

Various certification organizations may be involved in certifying products in accordance with NSF/ANSI 61. Individual states or local agencies have authority to accept

* Water Research Foundation, 6666 W. Quincy Avenue, Denver, CO 80235.

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

‡ NSF International, 789 N. Dixboro Road, Ann Arbor, MI 48105.

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

or accredit certification organizations within their jurisdiction. Accreditation of certification organizations may vary from jurisdiction to jurisdiction.

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

ANSI/AWWA 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 have been removed from the standard and need to be addressed by the purchaser.

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

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 Section 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.

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

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

(NFPA)* document NFPA 22, 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, Standard for 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.
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,

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

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 61, Drinking Water System Components—Health Effects, is required.

3. Capacity.

4. Bottom capacity level (BCL) or top capacity level (TCL) 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, or provincial and local requirements (Sec. 2.1).

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. Whether design snow loading may not be reduced if tank is located where the lowest one-day mean low temperature is 5°F (–15°C) or warmer (Sec. 3.1.3.1).

20. If tank is located in a special wind region, specify the basic wind speed (Sec. 3.1.4.1).

21. 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).

22. 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).

23. Location of manholes, ladders, and any additional accessories required (Section 5).

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

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

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

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

28. Whether safety cages, rest platforms, roof–ladder handrails, 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 8 ft (2.4 m) above grade (Sec. 5.4.2.2).

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

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

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

32. 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 3/8 in. (9.5 mm) (Sec. 8.4.2 (2)).

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

34. Whether the purchaser will provide shop inspection.

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

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

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

38. 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. (150 mm) above finish grade (Sec. 12.7.1).

39. 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.

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

* Occupational Safety and Health Administration, 200 Constitution Avenue N.W., Washington, DC 20210.

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

42. 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 4.

43. Seismic Use Group for the tank (Sec. 13.2.1).

44. Whether the site-specific procedure of Sec. 13.2.8 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 61, Drinking Water System Components—Health Effects, is required.

3. Capacity.

4. TCL 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 (Sec 2.1).

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. Whether design snow loading may not be reduced if tank is located where the lowest one-day mean low temperature is 5°F (–15°C) or warmer (Sec. 3.1.3.1).

18. If tank is located in a special wind region, specify the basic wind velocity (Sec. 3.1.4.1).

19. 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 Section 14.

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

21. Location of manholes, ladders, and additional accessories required (Section 7).

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

23. 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).

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

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

26. Whether safety cages, rest platforms, roof–ladder handrails, 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 8 ft (2.5 m) above the level of the tank bottom (Sec. 7.4.2.2).

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

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

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

30. For butt-joint welds subject to secondary stress, whether complete joint penetration is to be provided at joints in materials of thicknesses greater than 3/8 in. (9.5 mm) (Sec. 8.4.2 (2)). NOTE: For tanks that are to comply with Section 14, complete joint penetration is required for all butt-welded shell joints.

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

32. Whether the purchaser will provide shop inspection.

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

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

35. Kinds of paint or protective coatings and number of coats required for inside and outside surfaces except underside of bottom (see ANSI/AWWA D102).

36. 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. NOTE: Unless otherwise specified, the top of the foundation is to be a minimum of 6 in. (150 mm) above the finish grade (Sec. 12.7.1).

37. 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.

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

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

40. 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 4.

41. Seismic Use Group for the tank (Sec. 13.2.1).

42. Whether the site-specific procedure of Sec. 13.2.8 is required.

43. Whether seismic design of roof framing and columns is required (Sec. 13.5.4.5) and amount of live loads to be used.

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

45. Whether a certified welding inspector is required for Section 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 TCL, 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 Section 14 is used).

b. diameter, height to the TCL, 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).

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

2. Section 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/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.

3. Section 10 was revised to include an erection tolerance multiplier for elements of ground-supported flat-bottom tanks that are designed in accordance with Sec. 3.4.4 and subject to small compressive stresses.

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

5. Section 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.

6. Section 14 was revised to clarify the DMT-thickness requirement for Category 1 and Category 2 materials when impacts are provided.

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

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 the department at standards@awwa.org.

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ANSI/AWWA D100-11
(Revision of ANSI/AWWA D100-05)

American Water Works Association

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 cover concrete–steel composite tank construction.* 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.

* Refer to ANSI/AWWA D107, Composite Elevated Tanks for Water Storage.

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 top capacity level (TCL) and emptied to the bottom capacity level (BCL).

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 TCL 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 TCL, unless otherwise specified.

10.3 Top capacity level (TCL): The water level defined by the lip of the overflow.

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.

Details of all welded joints shall be provided when specified. Standard weld symbols as listed in ANSI/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* 301-05—Specifications for Structural Concrete.

ACI 318-08—Building Code Requirements for Structural Concrete.

AISC†—Steel Construction Manual, 13th Edition.

ANSI‡/AWS§ A2.4-07—Standard Symbols for Welding, Brazing, and Nondestructive Examination.

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

ANSI/AWS A5.1-04—Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding.

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

ANSI/AWS B2.1-09—Specification for Welding Procedure and Performance Qualification.

ANSI/AWS D1.1-08—Structural Welding Code—Steel, 2009 Errata.

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

ANSI/AWWA C652-02—Disinfection of Water-Storage Facilities.

ANSI/AWWA D102-06—Coating Steel Water-Storage Tanks.

ANSI/AWWA D108-10—Aluminum Dome Roofs for Water Storage Facilities.

API¶ 5L—Specification for Line Pipe, 44th Edition, February 2009 Addendum.

API 650—Welded Steel Tanks for Oil Storage, 11th Edition.

ASCE** 7-05—Minimum Design Loads for Buildings and Other Structures.

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

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

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

ASME* B16.5-09—Pipe Flanges & Flanged Fittings: NPS ½ Through NPS 24.

ASME BPVC Sec. V—Boiler and Pressure Vessel Code; Nondestructive Examination, 2007 Edition, 2009b Addenda.

ASME BPVC Sec. VIII, Div. 1—Boiler and Pressure Vessel Code; Rules for Construction of Pressure Vessels, 2007 Edition, 2009b Addenda.

ASME BPVC Sec. VIII, Div. 2—Boiler and Pressure Vessel Code; Alternative Rules; Rules for Construction of Pressure Vessels, 2007 Edition, 2009b Addenda.

ASME BPVC Sec. IX—Boiler and Pressure Vessel Code; Qualification Standard for Welding and Brazing Procedures, Welders, Brazers, and Welding and Brazing Operators, 2007 Edition, 2009b Addenda.

ASNT† SNT-TC-1A-06—Recommended Practice for Personnel Qualification and Certification in Nondestructive Testing.

ASTM‡ A6-09—Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling.

ASTM A20-09—Standard Specification for General Requirements for Steel Plates for Pressure Vessels.

ASTM A27-08—Standard Specification for Steel Castings, Carbon, for General Application.

ASTM A36-08—Standard Specification for Carbon Structural Steel.

ASTM A53-07—Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless.

ASTM A105-09—Standard Specification for Carbon Steel Forgings for Piping Applications.

ASTM A106-08—Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service.

ASTM A108-07—Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished.

ASTM A131-08—Standard Specification for Structural Steel for Ships.

ASTM A139-04—Standard Specification for Electric-Fusion (Arc)-Welded Steel Pipe (NPS 4 and over).

* ASME International, Three Park Avenue, New York, NY 10016.
† 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 A181-06—Standard Specification for Carbon Steel Forgings for General-Purpose Piping.

ASTM A193-09—Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High Temperature or High Pressure Service and Other Special Purpose Applications.

ASTM A283-03 (Reapproved 2007)—Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates.

ASTM A307-07b—Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength.

ASTM A325-09ae1—Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength.

ASTM A333-05—Standard Specification for Seamless and Welded Steel Pipe for Low-Temperature Service.

ASTM A350-07—Standard Specification for Carbon and Low-Alloy Steel Forgings, Requiring Notch Toughness Testing for Piping Components.

ASTM A370-09ae1—Standard Test Methods and Definitions for Mechanical Testing of Steel Products.

ASTM A435-90 (Reapproved 2007)—Standard Specification for Straight-Beam Ultrasonic Examination of Steel Plates.

ASTM A500-09—Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes.

ASTM A501-07—Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing.

ASTM A516-06—Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service.

ASTM A517-06—Standard Specification for Pressure Vessel Plates, Alloy Steel, High-Strength, Quenched and Tempered.

ASTM A524-96 (Reapproved 2005)—Standard Specification for Seamless Carbon Steel Pipe for Atmospheric and Lower Temperatures.

ASTM A537-08—Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel.

ASTM A568-09a—Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for.

ASTM A573-05 (Reapproved 2009)—Standard Specification for Structural Carbon Steel Plates of Improved Toughness.

ASTM A588-05—Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi (345 MPa) Minimum Yield Point, with Atmospheric Corrosion Resistance.

ASTM A592-04 (Reapproved 2009)—Standard Specification for High-Strength Quenched and Tempered Low-Alloy Steel Forged Fittings and Parts for Pressure Vessels.

ASTM A633-01 (Reapproved 2006)—Standard Specification for Normalized High-Strength Low-Alloy Structural Steel Plates.

ASTM A662-03 (Reapproved 2007)—Standard Specification for Pressure Vessel Plates, Carbon-Manganese-Silicon Steel, for Moderate and Lower Temperature Service.

ASTM A668-04—Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use.

ASTM A678-05 (Reapproved 2009)—Standard Specification for Quenched and Tempered Carbon and High-Strength Low-Alloy Structural Steel Plates.

ASTM A992-06a—Standard Specification for Structural Steel Shapes.

ASTM A1011-09b—Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, and High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength.

ASTM D1751-04 (Reapproved 2008)—Standard Specification for Preformed Expansion Joint Filler for Concrete Paving and Structural Construction (Non-extruding and Resilient Bituminous Types).

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

CSA* G40.21-04 (R2009)—General Requirements for Rolled or Welded Structural Quality Steels.

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

AISI‡ T-192—Steel Plate Engineering Data, Volumes 1 and 2, 1992 Edition.

* Canadian Standards Association, 178 Rexdale Blvd., Toronto, Ont., Canada M9W 1R3.

† Occupational Safety and Health Administration, 200 Constitution Ave., NW Washington, DC 20210.

‡ American Iron and Steel Institute, 1140 Connecticut Ave. NW, Suite 705, Washington, DC 20036.

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 Requirements

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

ASTM A193, grade B7 bolts and ASTM F1554, grade 105 bolts shall not be used unless mild steel anchor bolts (ASTM A307, grade B; ASTM A36; or ASTM F1554, grades 36 or 55) exceed 2½ in. (63 mm) in diameter. When ASTM F1554, grade 55 bolts are used, they shall comply with the weldable steel requirements of ASTM F1554, Supplement S1.

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; A131, grades A and B; A283, grades A, B, C, and D; or A573, grade 58; and materials listed in Sec. 2.2.3.2 and 2.2.7.

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

2.2.3.1.1 ASTM A36 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 shell plates greater than 1½ in. (38 mm) and less than or equal to 2 in. (51 mm) in thickness shall be killed. Plates

Table 1 Thickness limitations and special requirements

	Plate Thickness (*in.*)				
Shell Plates:	$t \leq ½$	$½ < t \leq 1$	$1 < t \leq 1½$	$1½ < t \leq 2$	$t > 2$
A36 (tension governs)	■	■	■	K, FGP	
A36 (compression governs)	■	■	■	K	K, FGP, N, UT
A573, Gr 58	■	■	■		
A131, Gr A	■				
A131, Gr B	■	■			
A283, Gr B, C (tension governs)	■	■			
A283, Gr B, C (compression governs)	■	■	■		
A283, Gr D	■	■ ($t \leq ¾$)			
Substitute material from Section 14 not listed above					
(tension governs)	■	■	■	■	
(compression governs)	■	■	■	■	■
Base Plates:					
A36/A283 Gr C	■	■	■	■	■
A36	■	■	■	■	■

■ Material may be used without special requirements

K = killed
FGP = fine-grain practice
N = normalized
UT = ultrasonic test

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.

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

2.2.3.1.3 ASTM A283, grade A steel is to be used only for nonstructural items such as clips, roof sheets, and other low-stressed components less than 1 in. (25 mm) thick. ASTM A283, grade B and 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, grade D shell plates are limited to a thickness of ¾ in. (19 mm).

2.2.3.1.4 ASTM A573, 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 or A283, grade C steels may be used for base plates regardless of thickness or temperature. A36 steel ordered as a bearing plate in accordance with ASTM A36, 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 Section 14 may be used for tanks designed in accordance with Section 3, without regard to thickness and temperature limitations of Section 14. Stress levels for substitute material shall be limited to those in Section 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.

2.2.4 *Sheets.* Sheet materials shall conform to ASTM A1011 SS, grade 30, 33, or 36, or ASTM A568. 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 or ASTM A992. 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.
2. Hot-formed tubing shall comply with ASTM A501.

2.2.5.2.1 Structural tubing with a circular cross section may be manufactured from plates of any of the specifications permitted in Sec. 2.2.3, provided 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 it complies with ASTM A139, grade B; ASTM A53 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. 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 G40.21 will have allowable design stresses per class 2 (see Section 3).

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

2.2.9 *Forgings.*

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

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, type E or S, grade B; ASTM A106; or API 5L or equal. Unless otherwise specified, joints may be screwed, flanged, or welded. Other pipe materials may be specified, provided 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:

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

3.1.1 *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.

3.1.2 *Water load.* Water load shall be the weight of all of the water when the tank is filled to the TCL. 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 foundations, shall not be considered a vertical load on the riser.

3.1.3 *Roof design loads.*

3.1.3.1 Snow load. The allowance for the pressure resulting from the design snow load shall be a minimum of 25 lb/ft^2 (1,205 N/m^2) on the horizontal projection of the tank and external balcony for roof surfaces having a slope of 30°, or less, with the horizontal. For roof surfaces with greater slope, the design-snow-load allowance shall be zero. The design-snow-load allowance may be reduced when the tank is located where the lowest one-day mean temperature is 5°F (–15°C), or warmer, and local experience indicates that a smaller load may be used.

3.1.3.2 Live load. The minimum roof design live load shall be 15 lb/ft^2 (720 N/m^2).

3.1.3.3 Deflection limit. There is no deflection limit for roof plates that span between structural supports.

3.1.4 *Wind load.* Wind pressure shall be calculated by the formula

$$P_w = q_z G C_f \geq 30 C_f \quad \text{(Eq 3-1)*}$$

Where:

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

G = gust-effect factor. The gust-effect factor shall be taken as 1.0 or may be calculated using the procedure given in ASCE 7. The calculated gust-effect factor shall be based on a damping ratio of 0.05 and shall not be less than 0.85.

C_f = force coefficient (see Table 2)

q_z = velocity pressure evaluated at height z of the centroid of the projected area, in pounds per square foot

$$q_z = 0.00256 K_z I V^2 \quad \text{(Eq 3-2)*}$$

* For equivalent metric equation, see Sec. 3.14.

Table 2 Force coefficient C_f

Type of Surface	C_f
Flat	1.0
Cylindrical or conical with apex angle* < 15°	0.60
Double curved or conical with apex angle ≥ 15°	0.50

*The apex angle is defined as the angle between the axis of revolution and the cone surface.

Table 3 Velocity pressure exposure coefficient K_z*

Height†		K_z	
ft	*(m)*	Exposure C	Exposure D
0 to 50	(15.2)	1.09	1.27
100	(30.5)	1.27	1.43
150	(45.7)	1.38	1.54
200	(61.0)	1.46	1.62
250	(76.2)	1.53	1.68
300	(91.4)	1.60	1.73
350	(106.7)	1.65	1.78

* K_z may be calculated in accordance with ASCE 7.
† Height above finish grade.

Where:

K_z = velocity pressure exposure coefficient evaluated at height z of the centroid of the projected area (see Table 3)

z = height above finished grade, in feet

I = wind importance factor = 1.15

V = basic wind speed, in miles per hour (see Figure 1, pages 14–18)

3.1.4.1 Basic wind speed. The basic wind speeds shown in Figure 1 are based on a 3-sec gust speed at 33 ft (10.1 m) above grade and an annual probability of 0.02 of being equaled or exceeded (50-year mean recurrence interval). In special wind regions, tanks may be exposed to wind speeds that exceed those shown in Figure 1. In such cases, the basic wind speed shall be specified.

3.1.4.2 Velocity pressure exposure coefficient. Velocity pressure exposure coefficients are provided for Exposure C and Exposure D in Table 3. Exposure C shall be used unless otherwise specified. The velocity pressure exposure coefficient shall be evaluated at height z of the centroid of the projected wind area. For intermediate heights, use linear interpolation or the larger of the velocity pressure exposure coefficients.

3.1.4.2.1 Surface roughness for Exposure C includes open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m). This category includes flat open country, grasslands, and all water surfaces in hurricane-prone regions.

3.1.4.2.2 Surface roughness for Exposure D includes flat, unobstructed areas and water surfaces outside hurricane-prone regions. This roughness includes smooth mud flats, salt flats, and unbroken ice. Exposure D shall apply where the aforementioned surface roughness prevails in the upwind direction for a distance greater than 5,000 ft (1,524 m) or 20 times the tank height, whichever is greater. Exposure D shall extend into downwind areas of surface roughness C for a distance of 600 ft (300 m) or 20 times the height of the tank, whichever is greater.

3.1.4.3 Columns, struts, and sway rods. For columns and struts of structural shapes, the projected area 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 wind pressure shall be applied on 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 tower, and the other on a vertical plane perpendicular to the first.

3.1.4.4 Wind pressures. The wind pressures defined by Eq 3-1 shall be applied to the projected areas of the tank, tank support structure, pilasters, and other ornamental features. The resulting wind loads shall be applied at the centroid of each area for the purpose of calculating overturning moments.

3.1.4.5 Shrouds. Where structures may be totally enclosed in a shroud for environmental protection during painting, the structure shall be checked for a wind velocity 50 percent greater than the maximum operating velocity of the shroud, but not more than 50 mph (22 m/sec) if the maximum operating velocity is unknown. The projected area of the shroud shall be the same height as the structure and 6 ft (1.8 m) wider than the projected area of the structure, unless otherwise specified.

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

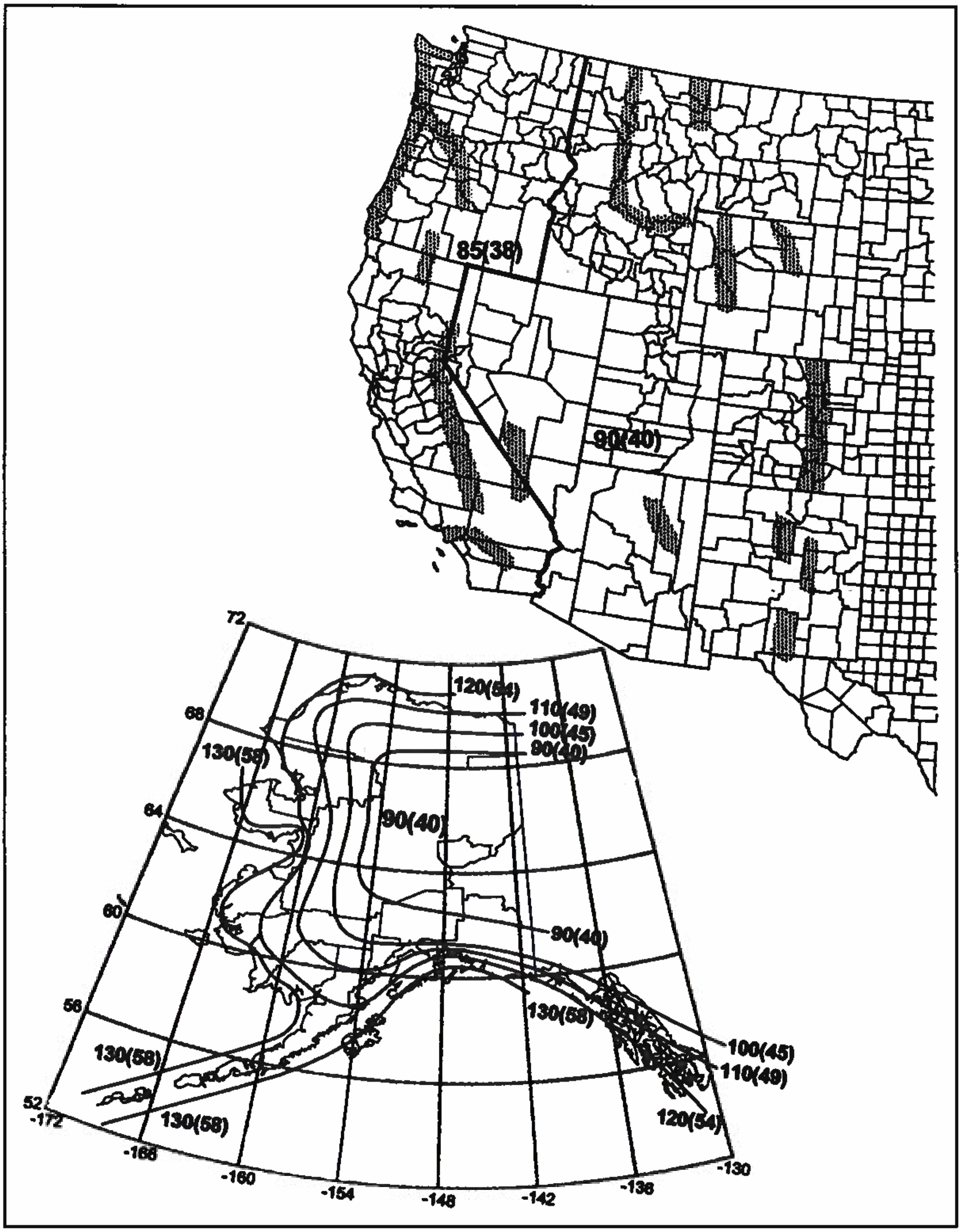

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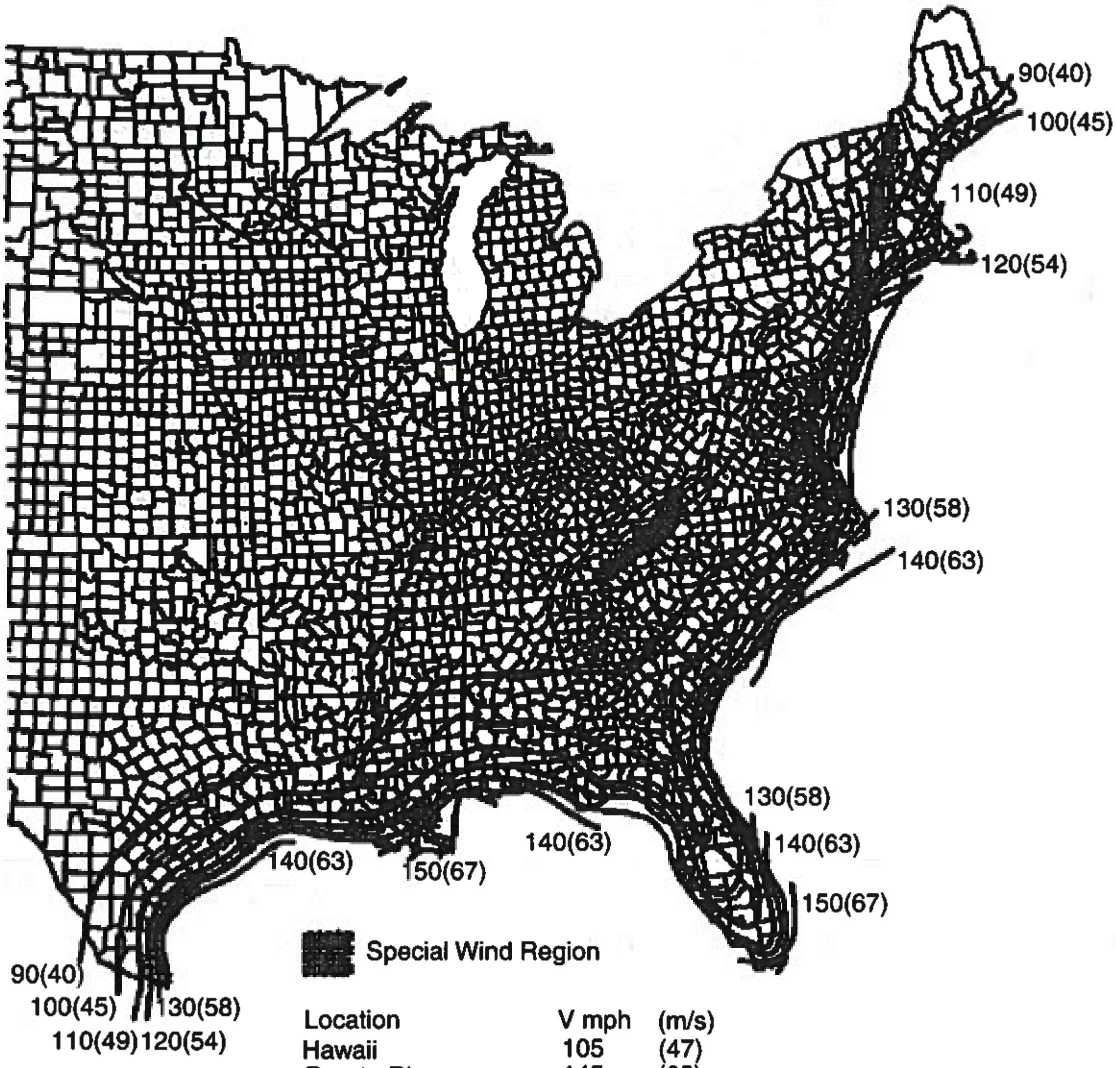
Figure 1 Basic wind speed V (from ASCE 7) (continued on next page)



Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	145	(65)
Guam	170	(76)
Virgin Islands	145	(65)
American Samoa	125	(56)

Notes:
1. Values are nominal design 3-sec gust wind speeds in miles per hour (m/s) at 33 ft (10 m) aboveground for Exposure C category.
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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Figure 1 Basic wind speed *V* (from ASCE 7) (continued on next page)

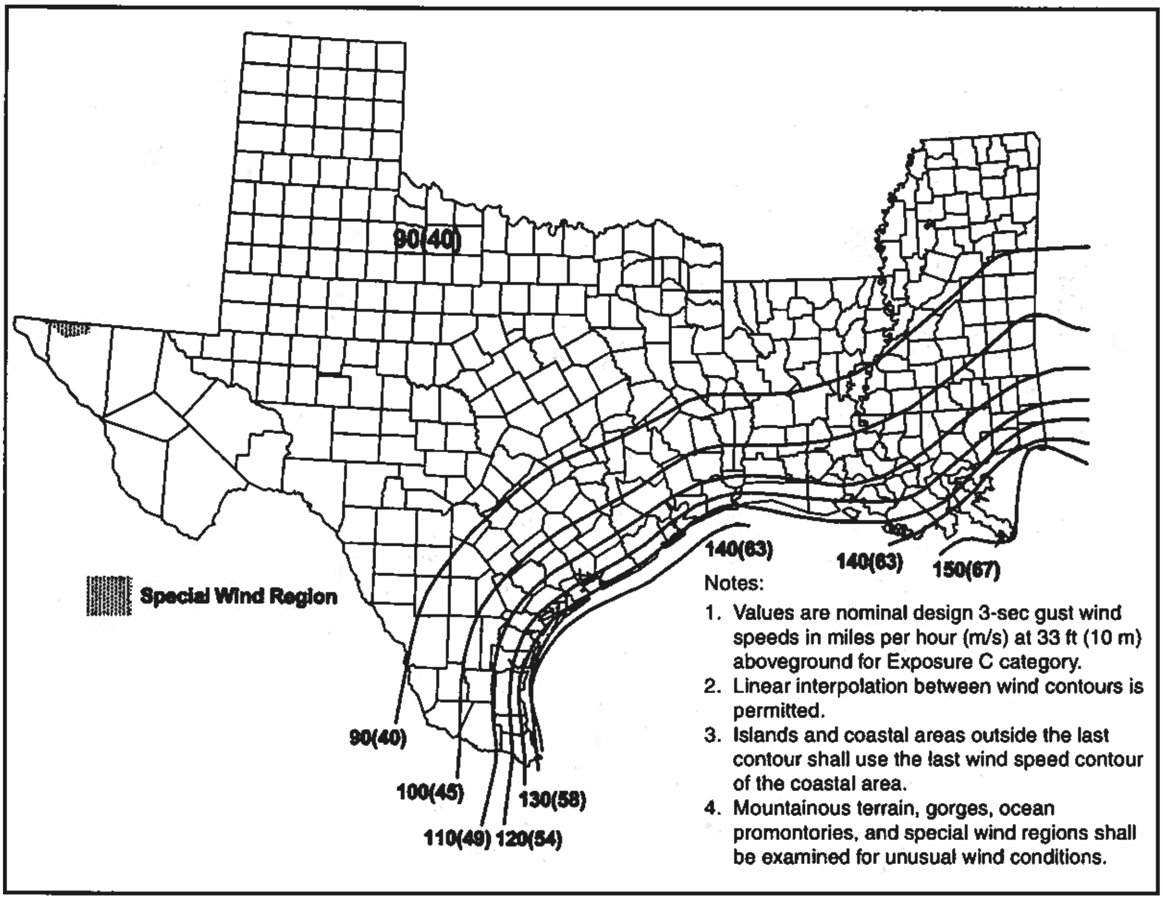

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Figure 1 Basic wind speed V (from ASCE 7) (continued on next page)

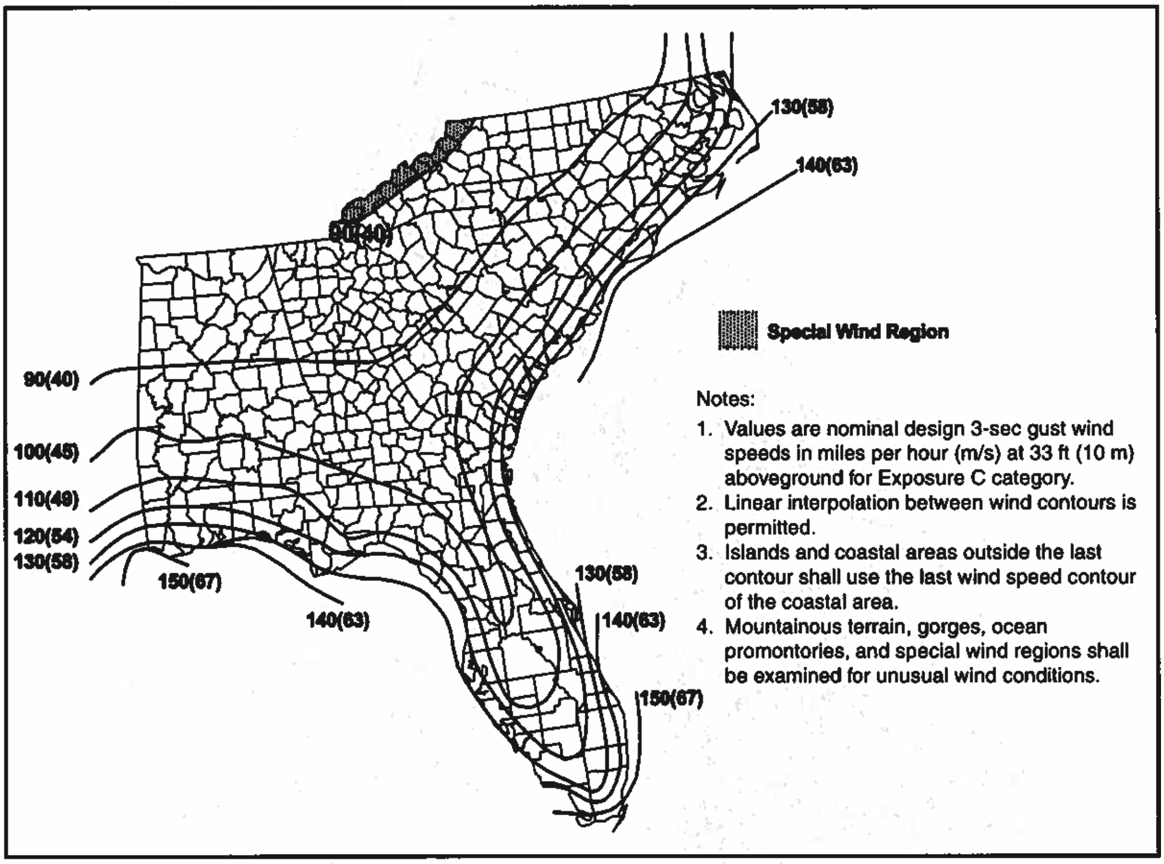

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Figure 1 Basic wind speed *V* (from ASCE 7) (continued on next page)

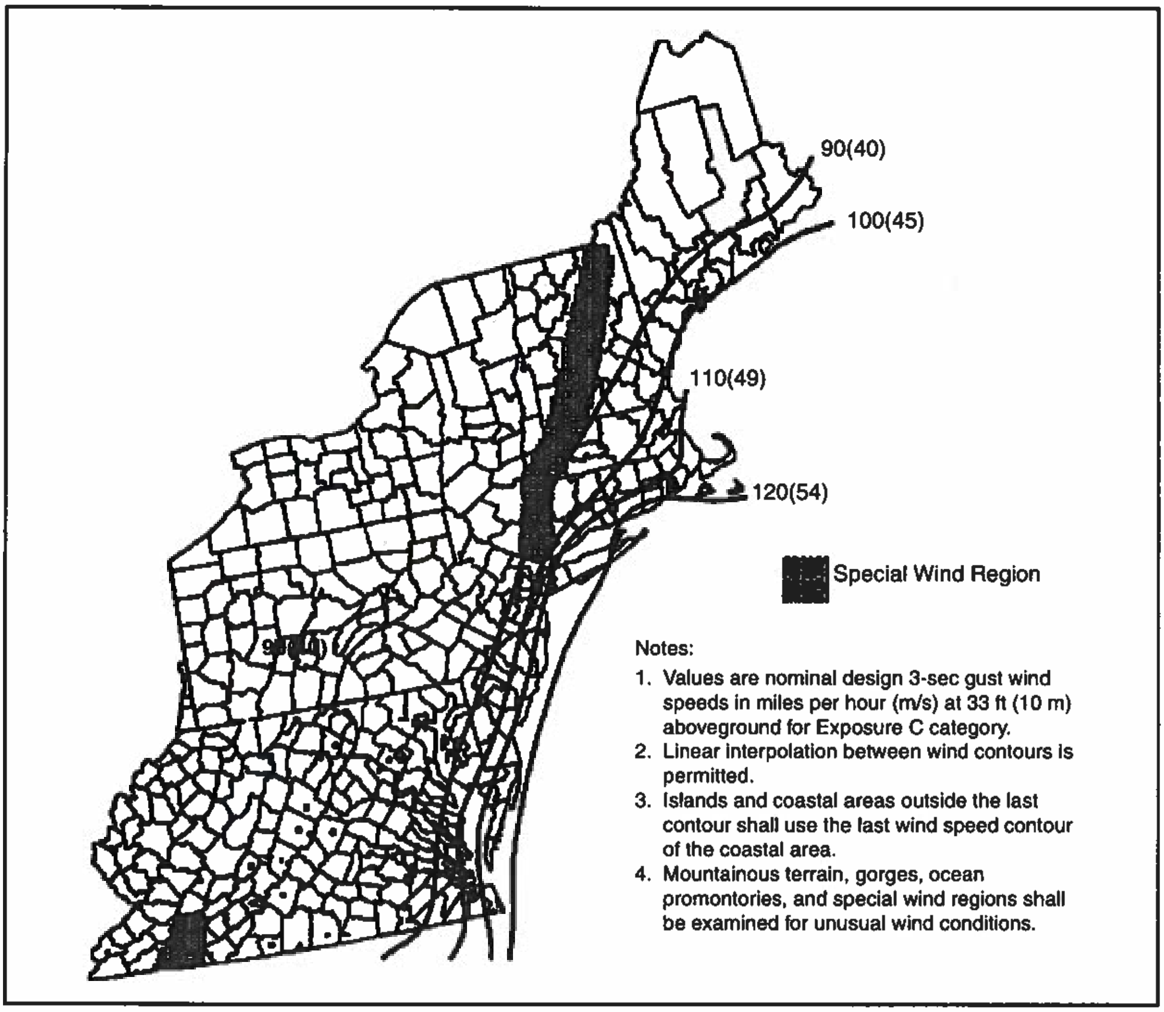

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Figure 1 Basic wind speed *V* (from ASCE 7)

3.1.5 *Seismic load.* Structures shall be designed for seismic loads as defined in Section 13. See Sec. 3.1.5.1 for exception.

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

3.1.5.2 Bracing details. For multicolumn tanks located where design for seismic loads is not required (Sec. 3.1.5.1), bracing that is part of the lateral force-resisting system shall be detailed to provide ductility in the event of an overload condition.

3.1.5.3 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.

3.1.5.4 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.

3.1.6 *Balcony, ladder, and stair 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, 500 lb (227 kg) to any 10-ft^2 (0.93-m^2) area on the tank roof, and 350 lb (159 kg) on each vertical section of the ladder. Structural parts and connections shall be proportioned properly to withstand such loads. The previously mentioned load need not be combined with the design snow load specified in Sec. 3.1.3, but it shall be combined with the dead load. The balcony, platform, and roof plating may deflect between structural supports in order to support the loading. Loading and design of stair systems shall comply with OSHA regulations.

3.1.7 *Handrail and guardrail assemblies.* Handrail and guardrail assemblies shall be designed in accordance with OSHA regulations. The assemblies shall be designed to resist a simultaneous vertical and horizontal load of 50 lb/ft (730 N/m) applied to the top rail and to transfer this load through supports to the structure. The horizontal load is to be applied perpendicular to the plane of the handrail or guardrail. Handrail and guardrail systems must be capable of withstanding a single concentrated load of 200 lb (890 N) applied in any direction at any point along the top and have attachment devices and the supporting structure to transfer this loading to appropriate structural elements. This load need not be considered to act concurrently with the previously specified 50-lb/ft (730-N/m) load. Intermediate rails shall be designed to withstand a horizontally applied normal load of 25 lb/ft (365 N/m).

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 three 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 15.

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, 3.4.3, 3.4.4, and 3.5, shall not exceed the limits shown for noncompact sections in AISC, Table B.4.1.

3.2.2 *Pipe thickness underrun.* Potential thickness underrun as permitted by the selected steel pipe requirement 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.

Table 4 Material classes

	F_y*	
Class	*psi*	*(MPa)*
0	$F_y < 27{,}000$	$(F_y < 186.2)$
1	$27{,}000 \le F_y \le 34{,}000$	$(186.2 \le F_y \le 234.4)$
2	$F_y > 34{,}000$	$(F_y > 234.4)$

*Where F_y is the published minimum yield strength.

Table 5 Unit stresses—tension

Item	Class	Maximum Unit Stress psi	(MPa)
Plates in tank shell	1,2*	15,000	(103.4)
Structural steel, built-up structural members, structural details	0	12,000	(82.7)
	1	15,000	(103.4)
	2	18,000	(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 or ASTM F1554-36		15,000	(103.4)
ASTM F1554-55 (weldable)		18,750	(129.3)
High-strength steel			
ASTM A193-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 having threads with root area greater than the rod area§	1	15,000	(103.4)
	2	18,000	(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.

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 3-3:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad \text{(Eq 3-3)}$$

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

(Eq 3-3 continued page 23)

Table 6 Unit stresses—compression

Item	Class	Maximum Unit Stress psi	(MPa)
Nonstructural items	0	12,000	(82.7)
Plates in tank shell, structural steel, built-up members, plate in structural applications, structural details, and weld metal	1	15,000	(103.4)
	2	18,000	(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	15,000	(103.4)
	2	18,000	(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	20,000	(137.9)
	2	24,000	(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* 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	0	9,600	(66.2
	1	12,000	(82.7)
	2	14,400	(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	30,000	(206.8)
	Single shear	24,000	(165.5)
Unfinished bolts	Double shear	18,750	(129.3)
	Single shear	15,000	(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.

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 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. 3.1.1, Sec. 3.1.2, and Sec. 3.1.3, 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 Section 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 Eq 3-4 through Eq 3-33 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 specified 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 = AISC effective column length factor

= 1.0 for pinned end columns or struts

= 2.0 for cantilever columns, such as the shaft of a single-pedestal tank

K_o = buckling coefficient

K_φ = slenderness reduction factor

K_1 = buckling coefficient

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)

$(t/R)_c$ = thickness-to-radius 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

$$F_a = F_L K_\varphi \qquad \text{(Eq 3-4)}$$

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

$$F_b = F_L \qquad \text{(Eq 3-5)}$$

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

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{f_a}{K_\varphi F_L} + \frac{f_b}{F_L} \leq 1.0 \qquad \text{(Eq 3-6)}$$

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 below:

When $$25 < \frac{KL}{r} \leq C'_c, \quad K_\varphi = 1 - \frac{1}{2}\left[\frac{\frac{KL}{r}}{C'_c}\right]^2 \qquad \text{(Eq 3-7)}$$

When $$\frac{KL}{r} \geq C'_c, \quad K_\varphi = \frac{1}{2}\left[\frac{C'_c}{\frac{KL}{r}}\right]^2 \qquad \text{(Eq 3-8)}$$

When $$\frac{KL}{r} \leq 25, \quad K_\varphi = 1.0 \qquad \text{(Eq 3-9)}$$

$$C'_c = \sqrt{\frac{\pi^2 E}{F_L}} \qquad \text{(Eq 3-10)}$$

3.4.2 *Structural sections.* The maximum permissible unit stress in compression for built-up and structural columns or struts shall be determined from Eq 3-4, Eq 3-5, and Eq 3-7 through Eq 3-10. 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 AISC, Table B4.1. When the ratio exceeds the limit, the allowable stress shall be reduced in accordance with AISC.

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.** Three methods are provided for calculating the allowable local buckling compressive stress F_L—Method 1, Method 2, and Method 3. The requirements for Methods 1, 2, and 3 are given in Sec. 3.4.3.1, Sec. 3.4.3.2, and Sec. 3.4.3.3, 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.

3.4.3.1.1 Class 1 materials. For class 1 materials, the thickness-to-radius ratio at which buckling changes from elastic to inelastic $(t/R)_c$ is 0.0031088. The allowable local buckling compressive stress for class 1 materials is given by the following formulas:

When $0 \le t/R \le (t/R)_c$, elastic buckling controls and

$$F_L = 17.5(10)^5\,(t/R)[1 + 50{,}000(t/R)^2] \qquad \text{(Eq 3-11)}^{\dagger}$$

---

* It is not the intent of this standard that this design method be used in the design of structures, such as composite tanks, which are beyond the scope of this standard.

† For equivalent metric equation, see Sec. 3.14.

When $(t/R)_c < t/R \leq 0.0125$, inelastic buckling controls and

$$F_L = 5{,}775 + 738(10)^3\ t/R \qquad \text{(Eq 3-12)*}$$

When $t/R > 0.0125$, plastic buckling controls and

$$F_L = 15{,}000 \text{ psi } (103.41 \text{ MPa})$$

3.4.3.1.2 Class 2 materials. For class 2 materials, the thickness-to-radius ratio at which buckling changes from elastic to inelastic $(t/R)_c$ is 0.0035372. The allowable local buckling compressive stress for class 2 materials is given by the following formulas:

When $0 \leq t/R \leq (t/R)_c$, elastic buckling controls and the allowable local buckling compressive stress is given by Eq 3-11.

When $(t/R)_c < t/R \leq 0.0125$, inelastic buckling controls and

$$F_L = 6{,}925 + 886(10)^3\ t/R \qquad \text{(Eq 3-13)*}$$

When $t/R > 0.0125$, plastic buckling controls and

$$F_L = 18{,}000 \text{ psi } (124.10 \text{ MPa})$$

3.4.3.1.3 Tables 10 through 13 have been generated based on the previous equations.

3.4.3.2 Method 2. Method 2 is a simplified design method that only applies to water-filled shells that meet the limitations of Sec. 3.4.3.2.1. Method 2 allows a partial increase in the allowable local buckling compressive stress caused by the stabilizing effect of hydrostatic pressure.

3.4.3.2.1 Method 2 only applies to water-filled cylinders and cones that meet the following requirements:

1. The plate thickness t determined by this method shall not be less than $0.9t_{base}$.
2. The water shall be contained within the shell such that hydrostatic pressure causes circumferential (hoop) tension stress in the shell.
3. The hydrostatic pressure p shall be 2 psi (0.0138 MPa) or greater.
4. The angle of the shell measured from the axis of revolution to the inside shell surface shall not exceed 55 degrees.
5. The shell elements shall be joined by complete joint penetration butt-joint welds. No lap-welded joints are permitted.
6. The thickness-to-radius ratio shall be greater than 0.0010, but less than or equal to 0.003. Method 1 (Sec. 3.4.3.1) shall be used for portions of the shell where the thickness-to-radius ratio exceeds 0.003.

* For equivalent metric equation, see Sec. 3.14.

Table 10 Allowable local buckling compressive stress F_L for class 1 materials

t/R	F_L *(psi)*	F_L *(MPa)*	*t/R*	F_L *(psi)*	F_L *(MPa)*	*t/R*	F_L *(psi)*	F_L *(MPa)*
0.0001	175	1.2	0.0043	8,943	61.7	0.0085	12,048	83.1
0.0002	351	2.4	0.0044	9,022	62.2	0.0086	12,122	83.6
0.0003	527	3.6	0.0045	9,096	62.7	0.0087	12,196	84.1
0.0004	706	4.9	0.0046	9,170	63.2	0.0088	12,259	84.6
0.0005	888	6.1	0.0047	9,244	63.7	0.0089	12,343	85.1
0.0006	1,069	7.4	0.0048	9,317	64.3	0.0090	12,417	85.6
0.0007	1,255	8.7	0.0049	9,391	64.8	0.0091	12,491	86.1
0.0008	1,445	10.0	0.0050	9,465	65.3	0.0092	12,565	86.7
0.0009	1,639	11.3	0.0051	9,539	65.8	0.0093	12,638	87.2
0.0010	1,838	12.7	0.0052	9,613	66.3	0.0094	12,712	87.7
0.0011	2,041	14.1	0.0053	9,686	66.8	0.0095	12,786	88.2
0.0012	2,251	15.5	0.0054	9,760	67.3	0.0096	12,860	88.7
0.0013	2,467	17.0	0.0055	9,834	67.8	0.0097	12,934	89.2
0.0014	2,690	18.6	0.0056	9,908	68.3	0.0098	13,007	89.7
0.0015	2,920	20.1	0.0057	9,982	68.8	0.0099	13,081	90.2
0.0016	3,158	21.3	0.0058	10,055	69.3	0.0100	13,155	90.7
0.0017	3,405	23.5	0.0059	10,129	69.9	0.0101	13,229	91.2
0.0018	3,660	25.2	0.0060	10,203	70.4	0.0102	13,303	91.7
0.0019	3,925	27.1	0.0061	10,277	70.9	0.0103	13,376	92.3
0.0020	4,200	29.0	0.0062	10,351	71.4	0.0104	13,450	92.8
0.0021	4,485	30.9	0.0063	10,424	71.9	0.0105	13,524	93.3
0.0022	4,782	33.0	0.0064	10,498	72.4	0.0106	13,598	93.8
0.0023	5,090	35.1	0.0065	10,572	72.9	0.0107	13,672	94.3
0.0024	5,410	37.3	0.0066	10,646	73.4	0.0108	13,745	94.8
0.0025	5,742	39.6	0.0067	10,720	73.9	0.0109	13,819	95.3
0.0026	6,088	42.0	0.0068	10,793	74.4	0.0110	13,893	95.8
0.0027	6,447	44.5	0.0069	10,887	74.9	0.0111	13,967	96.3
0.0028	6,821	47.0	0.0070	10,941	75.5	0.0112	14,041	96.8
0.0029	7,209	49.7	0.0071	11,015	76.0	0.0113	14,114	97.3
0.0030	7,612	52.5	0.0072	11,089	76.5	0.0114	14,188	97.8
0.0031	8,032	55.4	0.0073	11,152	77.0	0.0115	14,262	98.4
0.0032	8,137	56.1	0.0074	11,236	77.5	0.0116	14,336	98.9
0.0033	8,210	56.6	0.0075	11,310	78.0	0.0117	14,410	99.4
0.0034	8,284	57.1	0.0076	11,384	78.5	0.0118	14,483	99.9
0.0035	8,358	57.5	0.0077	11,453	79.0	0.0119	14,557	100.4
0.0036	8,432	58.2	0.0078	11,531	79.5	0.0120	14,631	100.9
0.0037	8,505	58.7	0.0079	11,585	80.0	0.0121	14,705	101.4
0.0038	8,579	59.2	0.0080	11,679	80.5	0.0122	14,779	101.9
0.0039	8,653	59.7	0.0081	11,753	81.1	0.0123	14,852	102.4
0.0040	8,727	60.2	0.0082	11,827	81.5	0.0124	14,926	102.9
0.0041	8,801	60.7	0.0083	11,900	82.1	0.0125	15,000	103.4
0.0042	8,875	61.2	0.0084	11,974	82.5	>0.0125	15,000	103.4

Table 11 Allowable local buckling compressive stress F_L for class 2 materials

t/R	F_L *(psi)*	F_L *(MPa)*	*t/R*	F_L *(psi)*	F_L *(MPa)*	*t/R*	F_L *(psi)*	F_L *(MPa)*
0.0001	175	1.2	0.0043	10,735	74.0	0.0085	14,456	99.7
0.0002	351	2.4	0.0044	10,823	74.6	0.0086	14,545	100.3
0.0003	527	3.6	0.0045	10,912	75.3	0.0087	14,633	100.9
0.0004	705	4.9	0.0046	11,001	75.9	0.0088	14,722	101.5
0.0005	886	6.1	0.0047	11,089	76.5	0.0089	14,810	102.1
0.0006	1,069	7.4	0.0048	11,178	77.1	0.0090	14,899	102.8
0.0007	1,265	8.7	0.0049	11,266	77.7	0.0091	14,988	103.4
0.0008	1,445	10.0	0.0050	11,355	78.3	0.0092	15,076	104.0
0.0009	1,639	11.3	0.0051	11,444	78.9	0.0093	15,165	104.5
0.0010	1,838	12.7	0.0052	11,532	79.5	0.0094	15,253	105.2
0.0011	2,041	14.1	0.0053	11,621	80.1	0.0095	15,342	105.8
0.0012	2,251	15.5	0.0054	11,709	80.8	0.0096	15,431	106.4
0.0013	2,467	17.0	0.0055	11,798	81.4	0.0097	15,519	107.0
0.0014	2,690	18.6	0.0056	11,887	82.0	0.0098	15,608	107.6
0.0015	2,920	20.1	0.0057	11,975	82.5	0.0099	15,696	108.3
0.0016	3,158	21.8	0.0058	12,064	83.2	0.0100	15,785	108.9
0.0017	3,405	23.5	0.0059	12,152	83.8	0.0101	15,877	109.5
0.0018	3,660	25.2	0.0060	12,241	84.4	0.0102	15,962	110.1
0.0019	3,925	27.1	0.0061	12,330	85.0	0.0103	16,051	110.7
0.0020	4,200	29.0	0.0062	12,418	85.6	0.0104	16,139	111.3
0.0021	4,485	30.9	0.0063	12,507	86.3	0.0105	16,228	111.5
0.0022	4,782	33.0	0.0064	12,595	86.9	0.0106	16,317	112.5
0.0023	5,090	35.1	0.0065	12,584	87.5	0.0107	16,405	113.1
0.0024	5,410	37.3	0.0066	12,773	88.1	0.0108	16,494	113.8
0.0025	5,742	39.6	0.0067	12,861	88.7	0.0109	16,582	114.4
0.0026	6,088	42.0	0.0068	12,960	89.3	0.0110	16,571	115.0
0.0027	6,447	44.5	0.0069	13,038	89.9	0.0111	16,760	115.6
0.0028	6,821	47.0	0.0070	13,127	90.5	0.0112	16,248	116.2
0.0029	7,209	49.7	0.0071	13,216	91.1	0.0113	16,937	116.8
0.0030	7,612	52.5	0.0072	13,304	91.8	0.0114	17,025	117.4
0.0031	8,032	55.4	0.0073	13,393	92.4	0.0115	17,114	118.0
0.0032	8,467	58.4	0.0074	13,481	93.0	0.0116	17,203	118.5
0.0033	8,919	61.5	0.0075	13,570	93.6	0.0117	17,291	119.2
0.0034	9,389	64.8	0.0076	13,659	94.2	0.0118	17,380	119.9
0.0035	9,877	68.1	0.0077	13,747	94.8	0.0119	17,468	120.5
0.0036	10,115	69.8	0.0078	13,836	95.4	0.0120	17,557	121.1
0.0037	10,203	70.4	0.0079	13,924	96.0	0.0121	17,646	121.7
0.0038	10,292	71.0	0.0080	14,013	96.6	0.0122	17,734	122.3
0.0039	10,380	71.6	0.0081	14,102	97.3	0.0123	17,823	122.9
0.0040	10,469	72.2	0.0082	14,190	97.9	0.0124	17,911	123.5
0.0041	10,558	72.8	0.0083	14,279	98.5	0.0125	18,000	124.1
0.0042	10,546	73.4	0.0084	14,367	99.1	>0.0125	18,000	124.1

Table 12 Allowable axial compressive stress F_a for combined effects of local buckling and slenderness for class 1 materials (psi)

t/R	KL/r = 25	KL/r = 50	KL/r = 75	KL/r = 100	KL/r = 125	KL/r = 150	KL/r = 175
0.0005	886	883	878	872	865	855	844
0.0010	1,838	1,823	1,804	1,779	1,745	1,705	1,657
0.0015	2,920	2,883	2,837	2,771	2,688	2,585	2,464
0.0020	4,200	4,123	4,027	3,892	3,719	3,507	3,256
0.0025	5,742	5,598	5,418	5,166	4,842	4,446	3,978
0.0030	7,613	7,359	7,043	6,600	6,031	5,335	4,512
0.0035	8,358	8,053	7,672	7,138	6,451	5,612	4,621
0.0040	8,727	8,394	7,979	7,397	6,648	5,733	4,652
0.0045	9,096	8,735	8,283	7,651	6,838	5,844	4,670
0.0050	9,465	9,074	8,585	7,900	7,020	5,944	4,673
0.0055	9,834	9,412	8,884	8,145	7,194	6,033	4,673
0.0060	10,203	9,748	9,180	8,384	7,361	6,111	4,673
0.0065	10,572	10,084	9,474	8,620	7,521	6,179	4,673
0.0070	10,941	10,418	9,765	8,850	7,674	6,236	4,673
0.0075	11,310	10,751	10,053	9,075	7,818	6,282	4,673
0.0080	11,679	11,083	10,339	9,296	7,956	6,318	4,673
0.0085	12,048	11,414	10,622	9,512	8,086	6,343	4,673
0.0090	12,417	11,744	10,902	9,724	8,209	6,357	4,673
0.0095	12,786	12,072	11,180	9,930	8,324	6,360	4,673
0.0100	13,155	12,399	11,455	10,132	8,431	6,360	4,673
0.0105	13,524	12,725	11,727	10,329	8,532	6,360	4,673
0.0110	13,893	13,050	11,996	10,521	8,625	6,360	4,673
0.0115	14,262	13,374	12,263	10,709	8,710	6,360	4,673
0.0120	14,631	13,696	12,527	10,891	8,788	6,360	4,673
0.0125	15,000	14,017	12,789	11,069	8,858	6,360	4,673

NOTES:
1. Interpolate for intermediate values.
2. To convert allowable stresses to MPa, divide values by 145.

Table 13 Allowable axial compressive stress F_a for combined effects of local buckling and slenderness for class 2 materials (psi)

t/R	*KL/r* = 25	*KL/r* = 50	*KL/r* = 75	*KL/r* = 100	*KL/r* = 125	*KL/r* = 150	*KL/r* = 175
0.0005	886	883	878	872	865	855	844
0.0010	1,838	1,823	1,804	1,779	1,745	1,705	1,657
0.0015	2,920	2,883	2,837	2,771	2,688	2,585	2,464
0.0020	4,200	4,123	4,027	3,892	3,719	3,507	3,256
0.0025	5,742	5,598	5,418	5,166	4,842	4,446	3,978
0.0030	7,613	7,359	7,043	6,600	6,031	5,335	4,512
0.0035	9,877	9,451	8,918	8,173	7,214	6,042	4,673
0.0040	10,469	9,990	9,392	8,554	7,477	6,161	4,673
0.0045	10,912	10,392	9,742	8,832	7,662	6,232	4,673
0.0050	11,355	10,792	10,088	9,103	7,836	6,287	4,673
0.0055	11,798	11,190	10,430	9,366	7,999	6,327	4,673
0.0060	12,241	11,587	10,769	9,623	8,151	6,351	4,673
0.0065	12,684	11,981	11,103	9,873	8,293	6,360	4,673
0.0070	13,127	12,374	11,434	10,117	8,423	6,360	4,673
0.0075	13,570	12,766	11,761	10,353	8,544	6,360	4,673
0.0080	14,013	13,155	12,083	10,583	8,653	6,360	4,673
0.0085	14,456	13,543	12,403	10,805	8,752	6,360	4,673
0.0090	14,899	13,930	12,718	11,021	8,840	6,360	4,673
0.0095	15,342	14,314	13,029	11,230	8,917	6,360	4,673
0.0100	15,785	14,697	13,337	11,432	8,984	6,360	4,673
0.0105	16,228	15,078	13,640	11,628	9,040	6,360	4,673
0.0110	16,671	15,457	13,940	11,816	9,085	6,360	4,673
0.0115	17,114	15,835	14,236	11,997	9,119	6,360	4,673
0.0120	17,557	16,211	14,528	12,172	9,143	6,360	4,673
0.0125	18,000	16,585	14,816	12,340	9,156	6,360	4,673

Notes:

1. Interpolate for intermediate values.
2. To convert allowable stresses to MPa, divide values by 145.

7. The material of construction shall have a minimum specified yield strength F_y equal to or greater than 36,000 psi (248.2 MPa).

8. The shell shall be uniformly supported at the lower boundary, similar to those found in single-pedestal elevated tanks.

9. Method 2 shall not be used adjacent to the lower boundary. Method 1 shall be used to determine the plate thickness within $4\sqrt{Rt}$ of the lower boundary.

3.4.3.2.2 The allowable local buckling compressive stress F_L shall be determined using Eq 3-14.

$$F_L = K_o 17.5(10)^5\,(t/R)[1 + 50{,}000(t/R)^2] \qquad \text{(Eq 3-14)*}$$

When 2 psi (0.0138 MPa) $\leq p <$ 10 psi (0.0689 MPa)

$$K_o = 1.0 + \left[\frac{0.25}{K_1 - 334}\right]\left[\frac{R}{t} - 334\right] \leq 1.25 \qquad \text{(Eq 3-15)}$$

$$K_1 = (950 - 50p) \qquad \text{(Eq 3-16)*}$$

When $p \geq$ 10 psi (0.0689 MPa)*

$$K_o = 1.0 + \left[\frac{0.25}{116}\right]\left[\frac{R}{t} - 334\right] \leq 1.25 \qquad \text{(Eq 3-17)}$$

3.4.3.3 Method 3. Method 3 is a complex design method based on a nonlinear buckling analysis. Method 3 only applies to water-filled shells that meet the limitations of Sec. 3.4.3.3.1. Method 3 allows an increase in the allowable local buckling compressive stress because of the stabilizing effect of hydrostatic pressure.

3.4.3.3.1 Method 3 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 thickness-to-radius ratios greater than or equal to 0.00143, and $0.7t_{base}$ for ratios less than 0.00143.

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 degrees.

4. The shell elements shall be joined by butt-welded joints with complete penetration. No lap-welded joints are permitted.

* For equivalent metric equation, see Sec. 3.14.

Table 14 Values of $(t/R)_c$

F_y *(psi)*	36,000	37,000	38,000	39,000	40,000
(MPa)	248.3	255.2	262.1	269.0	275.9
$(t/R)_c$	0.00299	0.00308	0.00316	0.00326	0.00334

5. The thickness-to-radius ratio shall be greater than 0.0010, but less than or equal to the value $(t/R)_c$ given in Table 14. Method 1 (Sec. 3.4.3.1) shall be used for portions of the shell where the thickness-to-radius ratio exceeds the Table 14 values.

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).

3.4.3.3.2 A nonlinear buckling analysis meeting the requirements of Sec. 3.4.3.3.3 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 Eq 3-18 and Eq 3-19:

$$F_L = \frac{F_{cr}}{2} \qquad \text{(Eq 3-18)}$$

$$F_L = (C_o + C_p)\frac{\eta E t}{2R} \qquad \text{(Eq 3-19)}$$

The elastic buckling coefficient is given by

$$C_o = \frac{102.2}{195 + \frac{R}{t}} \quad \text{for } t/R > 0.00161 \qquad \text{(Eq 3-20)}$$

$$C_o = 0.125 \quad \text{for } t/R \le 0.00161 \qquad \text{(Eq 3-21)}$$

The elastic buckling coefficient for pressure stabilization C_p shall be determined by the following equations:

$$C_p = \frac{1.06}{3.24 + \frac{Et}{f_b R}} \quad \text{for cylindrical and double-curved shells} \qquad \text{(Eq 3-22)}$$

$$C_p = \frac{1.33}{3.33 + \frac{Et}{f_b R}} \quad \text{for conical shells} \qquad \text{(Eq 3-23)}$$

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:

$$F_{eff} = \sqrt{F_{ch}^2 + F_{ch}F_h + f_h^2} \qquad \text{(Eq 3-24)}$$

$$\Delta = (C_o + C_p)\left[\frac{Et}{R}\right]\left[\frac{F_{eff}}{F_{ch}F_y}\right] \qquad \text{(Eq 3-25)}$$

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.

$$\eta = 1 \text{ for } \Delta \leq D_1 \qquad \text{(Eq 3-26)}$$

$$\eta = \frac{A}{B + \Delta^{0.80}} \text{ for } D_1 < \Delta < D_2 \qquad \text{(Eq 3-27)}$$

$$\eta = \frac{1}{\Delta} \text{ for } \Delta \geq D_2 \qquad \text{(Eq 3-28)*}$$

$$A = 0.651 + 3.83(10)^{-6}F_y \qquad \text{(Eq 3-29)}$$

$$B = 0.355 - 2.92(10)^{-6}F_y \qquad \text{(Eq 3-30)*}$$

$$D_1 = \frac{F_y}{136{,}000} + 0.196 \qquad \text{(Eq 3-31)*}$$

$$D_2 = \frac{118.34}{F_y^{0.0573}} - 60.17 \qquad \text{(Eq 3-32)*}$$

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

$$F_{ch} = (C_o + C_p)\frac{\eta Et}{R} \qquad \text{(Eq 3-33)}$$

5. If the compressive failure stress F_{ch} calculated in step 4 equals the compressive failure stress F_{ch} assumed in step 1, 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.

3.4.3.3.3 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.

* For equivalent metric equation, see Sec. 3.14.

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}.

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 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

$$l_e = \frac{b}{t} = \frac{7{,}300}{\sqrt{f}}\left(1.0 - \frac{1{,}590}{\left(\frac{w}{t}\right)\sqrt{f}}\right) \qquad \text{(Eq 3-34)*}$$

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: w shall not be less than the work point width less $6t$.

* For equivalent metric equation, see Sec. 3.14.

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.

3.4.4.1 Width-to-thickness limits. Flat-plate elements other than those addressed by Sec. 3.4.4 shall be designed using the allowable stress design provisions of AISC.

Sec. 3.5 Shell Girder, Intermediate Stiffeners, and Compression Rings

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

$$S = 0.0001HD^2\left(\frac{P_{aw}}{18}\right) \qquad \text{(Eq 3-35)*}$$

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 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, as calculated by Eq 3-1. 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

3.5.1.1 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.

3.5.2 *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

$$h = \frac{10.625 \times 10^6 t}{P_{aw}\left(\frac{D}{t}\right)^{1.5}} \qquad \text{(Eq 3-36)*}$$

* For equivalent metric equation, see Sec. 3.14.

Where:

h = the height of the cylindrical shell between the intermediate wind girder and the roof, top angle, or top wind girder, in feet (NOTE: Where an ornamental roof [torus] transition exists, two-thirds of the transition height shall be added to the shell height. If the torus is stiffened by radial stiffeners at a spacing of 7 ft [2.31 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 stiffening or the top of the straight cylindrical portion of the shell, whichever is greater.)

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, as calculated per Eq 3-1. For intermediate shell girders, the design height shall be taken as h.

3.5.2.1 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 stiffeners, 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

$$W_{tr} = W\left(\frac{t_{uniform}}{t_{actual}}\right)^{2.5} \qquad \text{(Eq 3-37)}$$

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 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.

3.5.2.2 Section required. When intermediate stiffeners are required, they shall be proportioned in accordance with the formula

$$S = 0.0001hD^2\left(\frac{P_{aw}}{18}\right) \qquad \text{(Eq 3-38)*}$$

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.

3.5.3 *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 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.

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 AISC with the following stipulations or exceptions:

* For equivalent metric equation, see Sec. 3.14.

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. When bracing is required to provide the necessary lateral support, bracing shall conform to the requirements of AISC. Where the roof plate is incorporated into built-up rafter sections, the roof plate may be considered effective for a distance of 16*t* 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 AISC. Columns subject to lateral loads shall be designed as beam-columns.

3.6.1.4 Rafter design. Roof rafters shall be designed using the allowable stress design provisions of AISC for A36 material when the roof design live load is 50 lb/ft^2 (2,400 N/m^2) or less. For roof design live loads greater than 50 lb/ft^2 (2,400 N/m^2), roof rafter design may utilize higher allowable stresses when using material with minimum specified yield strength greater than A36 material. 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 top capacity level. No part shall project below the top capacity level.

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 rafter spacing. Maximum rafter spacing for supported roofs shall be

$$L = \frac{2{,}575t}{\sqrt{W_{D+L}}} \leq 84 \qquad \text{(Eq 3-39)*}$$

Where:

t = roof plate thickness, in inches

L = rafter centerline spacing at maximum radius, in inches

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

* For equivalent metric equation, see Sec. 3.14.

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 prior to 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.

3.6.2 *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. 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

$$t = \frac{2.6 h_p D G}{sE} \qquad \text{(Eq 3-40)*}$$

Where:

t = the required design shell plate thickness, in inches

h_p = the height of liquid from TCL 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 15)

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

* For equivalent metric equation, see Sec. 3.14.

Table 15 Weld design values—tank plate joints

Type of Joint	Efficiency-percent Tension	Efficiency-percent 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 joint edge	$75\frac{(1+X)}{2}$†	$75\frac{(1+X)}{2}$†
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 thickness) 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).

seismic loads exceed the limits for self-anchored tanks. Mechanical anchorage shall always be provided for elevated tanks.

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

3.8.3 *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.

3.8.4 *Unit stresses.*

3.8.4.1 Static loads. The allowable unit tension stress for ASTM A36, ASTM F1554, and ASTM A193, 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.

3.8.4.2 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.

3.8.4.3 Seismic loads. The allowable unit tension stress for seismic loads for ASTM A36, ASTM F1554-36, and ASTM F1554-55 (weldable) anchor bolts shall be as given in Sec. 3.3.3.2. For ASTM A193, 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.

3.8.4.4 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.

3.8.5 *Anchor requirements.*

3.8.5.1 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.

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 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.

3.8.5.2 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.

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 degrees.

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.

3.8.6 *Design of anchor chairs.*

3.8.6.1 Anchor-bolt chairs. Anchor-bolt chairs shall be designed in accordance with AISI T-192. Other design procedures may be used, provided they are based on a detailed analysis (e.g., finite element analysis) and account for local shell stresses. For designs in accordance with AISI T-192, the allowable local shell or pedestal stress shall be 20,000 psi (137.9 MPa) plus the permissible increase for wind or seismic loads, if applicable. For designs based on a detailed analysis, local shell or pedestal stress shall be evaluated using a procedure similar to that given in Appendix 4 of ASME BPVC Sec. VIII, Div. 2, with S_m taken as the allowable tensile stress defined by this section.

3.8.6.2 Alternate configurations. Alternate anchor-chair configurations may be used, provided that they are proven by test or calculation to be comparable in strength to the above and that stresses of all components are limited to those specified in Sec. 3.2 and, if applicable, increased per Sec. 3.3.3.

3.8.7 *Design for resistance to base shear.*

3.8.7.1 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. 13.5.4.6. V_{NET} shall be resisted by only that portion of the shear anchorage system considered to be effective for any direction of ground motion.

3.8.7.2 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.

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

1. Design of anchor embedment shall be in accordance with UBC, ACI 318, or ACI 349.
2. At least 3 in. (76.2 mm) clear cover between the anchor and bottom of the foundation shall be provided.
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.

3.8.9 *Design loads.*

3.8.9.1 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. When the calculated net uplift force for both P_W and P_S results in a negative value, no uplift anchorage is required. For single-pedestal and ground-supported flat-bottom tanks, the design uplift forces P_W and P_S are calculated as follows:

$$P_W = \frac{4M_W}{ND_{ac}} - \frac{W'}{N} \quad \text{(Eq 3-41)*}$$

$$P_S = \frac{4M_S}{ND_{ac}} - \frac{W'}{N} \quad \text{(Eq 3-42)*}$$

Where:

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

* For equivalent metric equation, see Sec. 3.14.

M_W, M_S = wind and seismic overturning moment, respectively, in foot-pounds

N = number of anchors

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

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

3.8.9.2 Anchor chairs and anchor attachments. Anchor chairs and anchor attachments to the tank shell, pedestal, or columns shall be designed for

1. P_W using allowable stresses of Sec. 3.2.
2. Lesser of $[16M_S/ND_{ac}] - W'/N$ and anchor yield capacity based on the corroded area at the root of the threads for anchor bolts or the corroded thickness for anchor straps. Anchor chairs and anchor attachments shall not fail for this load.

3.8.9.3 Special. See Sec. 13.3.1 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. 3.8.5.

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.

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 TCL 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

Table 16 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		Other Tanks	
ft	*(m)*	*ft*	*(m)*	*in.*	*(mm)*	*in.*	*(mm)*
$D \leq 20$	$D \leq 6.1$	All	All	3/16	4.76	1/4	6.35
$20 < D \leq 50$	$6.1 < D \leq 15.2$	$H \leq 48$	$H \leq 14.6$	3/16	4.76	1/4	6.35
$20 < D \leq 50$	$6.1 < D \leq 15.2$	$H > 48$	$H > 14.6$	1/4	6.35	1/4	6.35
$50 < D \leq 120$	$15.2 < D \leq 36.6$	All	All	1/4	6.35	1/4	6.35
$120 < D \leq 200$	$36.6 < D \leq 61.0$	All	All	5/16	7.94	5/16	7.94
$D > 200$	$D > 61.0$	All	All	3/8	9.52	3/8	9.52

thickness of double butt-welded 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 TCL, 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 1/4 in. (6.35 mm).

3.10.3 *Cylindrical shell plates.* Cylindrical shell plates in contact with water when the tank is filled to the TCL shall have minimum thicknesses as shown in Table 16.

3.10.4 *Brace rods.* Bars used for wind bracing shall have a minimum diameter or width of 3/4 in. (19 mm). Other shapes used for wind bracing shall have a total net cross-sectional area at least equal to that of a 3/4-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 TCL shall have a minimum thickness of 3/16 in. (4.76 mm).

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. 3.8.5.

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 ¼-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 15.

Sec. 3.13 Reinforcement Around Openings

Shell cutouts 4 in. (102 mm) in diameter and less 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 above.

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 Section 14, category 3.

2. Cleanouts for tanks built in accordance with the design criteria of Section 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 number 3 below, and shall be constructed of material from category 1 or 2 of Section 14.

3. Cleanouts for tanks conforming to Section 3 that exceed the limits of number 2 above, and cleanouts for tanks conforming to Section 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 Section 3 are as follows:

Equation Number	Equivalent Metric Equation	Variable	Units
3-1	$P_w = q_z G C_f \geq 1{,}436 C_f$	P_w, q_z	N/m²
3-2	$q_z = 0.613 K_z I V^2$	q_s V	N/m² m/s
3-11	$F_L = 12{,}066\left(\frac{t}{R}\right)\left[1 + 50{,}000\left(\frac{t}{R}\right)^2\right]$	F_L t,R	MPa mm
3-12	$F_L = 39.82 + 5{,}089\left(\frac{t}{R}\right)$	F_L t,R	MPa mm
3-13	$F_L = 47.75 + 6{,}109\left(\frac{t}{R}\right)$	F_L t,R	MPa mm
3-14	$F_L = K_o 12{,}066\left(\frac{t}{R}\right)\left[1 + 50{,}000\left(\frac{t}{R}\right)^2\right]$	F_L t,R	MPa mm
3-16	$K_1 = 950 - 7{,}252p$	p	MPa
3-29	$A = 0.651 + 0.000555F_y$	F_y	MPa
3-30	$B = 0.355 - 0.000424F_y$	F_y	MPa
3-31	$D_1 = \frac{F_y}{937.69} + 0.196$	F_y	MPa
3-32	$D_2 = \frac{88.98}{F_y^{0.0573}} - 60.17$	F_y	MPa
3-34	$l_e = \frac{b}{t} = \frac{606}{\sqrt{f}}\left(1.0 - \frac{132}{\left(\frac{w}{t}\right)\sqrt{f}}\right)$	f w,b,t	MPa mm
3-35	$S = 0.06713 H D^2 P_{aw}$	S H,D P_{aw}	mm³ m N/m²
3-36	$h = \frac{8{,}025t}{P_{aw}\left(\frac{D}{t}\right)^{1.5}}$	h,D P_{aw} t	m N/m² mm

3-38	$S = 0.06713hD^2P_{aw}$	S h,D P_{aw}	mm^3 m N/m^2
3-39	$L = \dfrac{17.8t}{\sqrt{W_{D+L}}} \le 2.13$	L t W_{D+L}	m mm N/m^2
3-40	$t = \dfrac{4.9h_pDG}{sE}$	t hp,D s	mm m MPa
3-41	$P_W = \dfrac{4M_W}{ND_{ac}} - \dfrac{9.81W'}{N}$	P_W M_W D_{ac} W'	N N-m m kg
3-42	$P_S = \dfrac{4M_S}{ND_{ac}} - \dfrac{9.81W'}{N}$	P_S M_S D_{ac} W'	N N-m m 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 17.

Table 17 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 bottom capacity level or to the top capacity level. 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 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, not 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 Section 3. The maximum unit stress shall be reduced for the joint efficiencies set forth in Table 15. 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 15 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. or less in width, sizes of openings when measured at the top of the opening shall be limited to 20° 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. 3.8.3, 12.4, and 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. 3.1.6. 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 nonload 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 safety cages, rest platforms, roof handrails, 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, spaced 12 in. (305 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 8 ft (2.4 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 5 in 12 or greater, a ladder or stairs shall be provided.
2. Where roof slope is less than 5 in 12 and greater than 2 in 12, a single handrail and nonskid surface shall be provided.
3. Where roof slope is 2 in 12 or less, handrails and nonskid surface are not required.

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 TCL. An opening shall be provided above the TCL. 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 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 TCL.

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 TCL, 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 be developed. 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, aluminum, etc.) installed inside the tank below the TCL shall be electrically isolated from the carbon steel tank components to which they are attached. Painting of the dissimilar metals does not eliminate the requirement for isolation.

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. 13.6.)

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 adequate provisions are made to protect the pipe from freezing and provided 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 Section 13 for potential seismic movements.

7.2.3 *Bottom connections.* Bottom connections shall comply with Sec. 13.6.2 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 3/8 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 safety cages, rest platforms, roof handrails, 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. × 3/8 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 3/4 in. (19 mm) round or square, spaced 12 in. (305 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 8 ft (2.4 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 5 in 12 or greater, a ladder or stairs shall be provided.
2. Where roof slope is less than 5 in 12 and greater than 2 in 12, a single handrail and nonskid surface shall be provided.
3. Where roof slope is 2 in 12 or less, handrails and nonskid surface are not required.

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 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 TCL, 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 be developed. 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, Commentary for Welded Carbon Steel Tanks for Water Storage, Sec. A.5.6.

Sec. 7.7 Galvanic Corrosion

Dissimilar metals (e.g., stainless steel, aluminum, etc.) installed inside the tank below the TCL shall be electrically isolated from carbon steel tank components to which they are attached. Painting of the dissimilar metals does not eliminate the requirement for isolation.

SECTION 8: WELDING

Sec. 8.1 Definitions and Symbols

Welding terms used in this standard shall be interpreted according to the definitions given in ANSI/AWS A3.0. Symbols used on construction drawings shall conform to those shown in ANSI/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 or ANSI/AWS B2.1. 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 they meet requirements of Annex III of ANSI/AWS D1.1. Impact testing, when required, shall be conducted at the low one-day mean temperature (see Figure 23) 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 in accordance with the rules in ASME BPVC Sec. IX, or, alternatively, to ANSI/AWS B2.1. 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. Where an AWS standard welding procedure is selected and can meet the specified requirement, qualification testing of that procedure is not necessary, provided that all of the rules in ASME BPVC Sec. IX or ANSI/AWS B2.1 that govern the use of such procedure are followed.

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 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, shall be accepted in P-Number 1, group 1, 2, or 3 material grouping of ASME BPVC Sec. IX. ASTM A517 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 described in ASME BPVC Sec. IX, or alternatively ANSI/AWS B2.1.

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 3/8 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 3/8 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 3/8 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 18, 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 18 Minimum size of fillet weld—shell-to-bottom joint

Thickness of Shell Plate					
Minimum		Maximum		Minimum Size of Fillet Weld	
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)	3/8	(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 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 the total depth of weld, including not more than $\frac{1}{16}$ 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 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.

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* 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 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 the ASME BPVC Sec. IX, and Sec. 10.3 and 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. (4.8 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.

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 19 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 19 and for the plate thicknesses given therein.

Table 19 Minimum diameter for unrolled shell plates for elevated tanks

Plate Thickness					
Minimum		Maximum		Minimum Diameter	
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 Shop Assembly

Double-curved tank bottoms, shells, and roofs shall be assembled in the shop, if necessary, to ensure that they will fit properly in the field.

Sec. 9.8 High-Strength Anchor Bolts

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

Sec. 9.9 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.

Table 20 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)

10.1.2 *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.

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 20. 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 below, 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), 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 ANSI/AWS A5.1 or ANSI/AWS A5.5, 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 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 or in ASTM A20, 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 21.

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.

Table 21 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)

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).

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 and A20, 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:

$$e_x = 0.01L_x \qquad \text{(Eq 10-1)}$$

$$L_x = 4\sqrt{Rt} \qquad \text{(Eq 10-2)}$$

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 equations 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}.

$$\text{For } C_{1cr} \geq 0.75,\ K_{1cr} = 1.0 \qquad \text{(Eq 10-3)}$$

$$\text{For } 0.75 > C_{1cr} \geq 0.25,\ K_{1cr} = 1.0 + \frac{0.75 - C_{1cr}}{0.50} \qquad \text{(Eq 10-4)}$$

$$\text{For } C_{1cr} < 0.25,\ K_{1cr} = 2.0 \qquad \text{(Eq 10-5)}$$

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 22 are not exceeded in the welded joint.

10.7.3 *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 they have a surface suitable for the subsequent cleaning and painting operations.

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.

Table 22 Maximum allowable offset of aligned butt joints

Thickness		Subject to Primary Stress		Subject to Secondary Stress	
in.	*(mm)*	*in.*	*(mm)*	*in.*	*(mm)*
$0 < t^* \leq 5/8$	$0 < t \leq 16$	1/16	1.6	1/8	3.2
$t > 5/8$	$t > 16$	Lesser of $0.10t$ or 1/4	Lesser of $0.10t$ or 6	Lesser of $0.20t$ or 3/8	Lesser of $0.20t$ or 9.5

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

Sec. 10.9 High-Strength Anchors

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

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 tank shell joints, particularly those subject to primary stress from weight or pressure of tank contents and load-bearing risers in contact with the water. 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 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 not subject to the weight or pressure of the tank contents.

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 20.

7. Fillet weld convexity in excess of

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

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

c. $3/16$ in. (4.8 mm) for width of individual bead or weld face 1 in. (25.4 mm) or over.

8. Undercut in excess of the limits given in Sec. 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 22.

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 21.

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) or Method 3 (see Sec. 3.4.3.3) 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 nonconforming 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 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 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 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 ten 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 $4/3$ times the published minimum yield strength. For each welder of this joint, testing shall be as follows:

11.4.5.1 Qualification testing. Prior to 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 and shall be made as soon as possible 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 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 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

Table 23 Maximum height of weld reinforcement of weld for butt joints above plate surface

Plate Thickness					
Minimum		Maximum		Maximum Height of Crown	
in.	*(mm)*	*in.*	*(mm)*	*in.*	*(mm)*
		½	(13)	1/16	(1.6)
> ½	(> 13)	1	(25)	3/32	(2.4)
> 1	(> 25)			⅛	(3.2)

accordance with the current edition of ASNT SNT-TC-1A and all supplements, NDT Level II.

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 23.

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 prior to 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* (*T* being the thickness of the thinner plate joined) in a length of 6*T*, 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 6*T*, the permissible sum of the lengths of all inclusions shall be proportionately less than *T*, provided 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 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 TCL. 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. 3.1.2, 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. 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. 13.2.4.

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.

12.3.1 *Cross-braced multicolumn tanks.* A safety factor of 3.0 shall be provided based on calculated ultimate bearing capacity for gravity loads.

12.3.1.1 Gravity loads plus wind load. A safety factor of 3.0 shall be provided based on calculated ultimate bearing capacity for gravity loads plus wind load, excluding overturning toe pressure caused by shear at the top of footing, unless otherwise specified. The safety factor may be reduced to 2.25 when specified in the geotechnical report.

12.3.1.2 Gravity loads plus seismic load. A safety factor of 2.25 shall be provided based on calculated ultimate bearing capacity for gravity loads plus seismic load, excluding overturning toe pressure caused by shear at the top of footing.

12.3.1.3 Overturning toe pressure. 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.

12.3.2 *Single-pedestal tanks, standpipes, and reservoirs.* A safety factor of 3.0 shall be provided based on calculated ultimate bearing capacity for gravity loads.

12.3.2.1 Gravity loads plus seismic load. A safety factor of 3.0 shall be provided based on calculated ultimate bearing capacity for gravity loads plus wind load, unless otherwise specified. The safety factor may be reduced to 2.25 when specified in the geotechnical report.

12.3.2.2 Gravity loads plus seismic load. A safety factor of 2.25 shall be provided based on calculated ultimate bearing capacity for gravity loads plus seismic load.

12.3.3 *Driven-pile foundations.* When a pile foundation is provided, a minimum safety factor of 2.0 shall be provided for gravity loads. A safety factor of 1.5 shall be provided for gravity loads plus wind or seismic loads.

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, per Sec. 12.3.1.1 and Sec. 12.3.1.2. Peak toe pressure caused by shear at the top of the foundation shall be combined with gravity loads and wind or seismic loads, per Sec. 12.3.1.3.

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 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. 12.5 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, per Sec. 12.3.2.1 and Sec. 12.3.2.2.

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 subjected to an overturning moment equal to 1.5 times the overturning moment determined for wind or seismic loads (see Figure 2).

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 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. 12.6 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 Section 14 shall be supported only on type 1 or type 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 cushion of compacted crushed stone, fine 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 (90 L) of heavy-base petroleum oil/yd^3 (89 L/m^3) of sand. 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. The tank cushion inside the ringwall shall be at least 3 in. (76 mm) thick and shall be provided above the earthen interior under the tank bottom. 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. A sand cushion not less than 1-in. (25-mm) thick shall be provided between the flat bottom and the concrete slab foundation. In lieu of a 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.

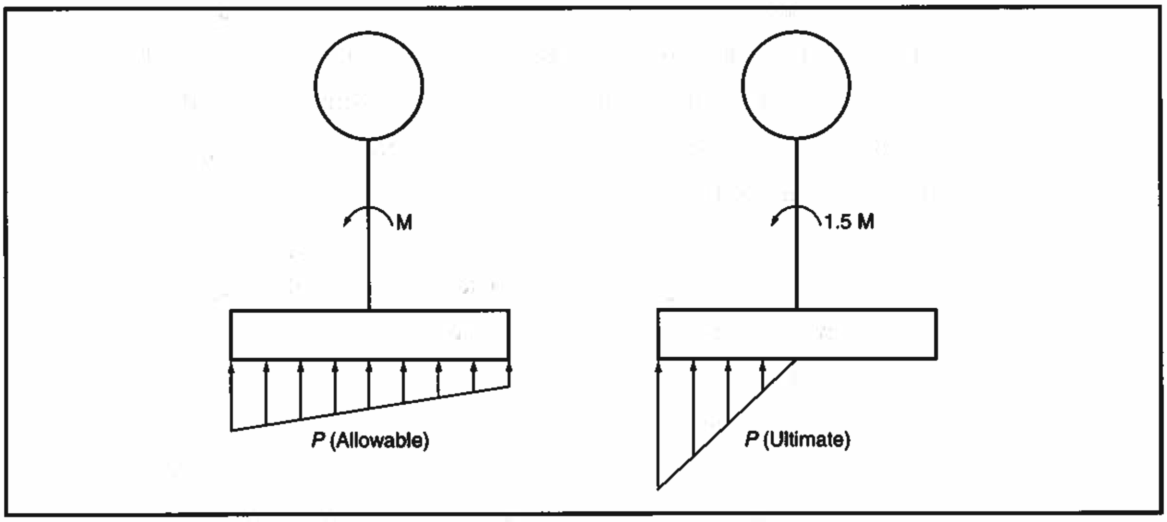

Figure 2 Diagram for checking overturning stability of pedestal-type elevated tanks (wind or seismic events)

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 a sand cushion within a concrete ringwall. The cushion base shall consist of a minimum of 6-in. (152-mm) cushion of clean sand or fine crushed stone saturated with a heavy-base petroleum oil. The top of the sand within the ringwall should slope uniformly upward from the top of the wall to the center of the tank. The inside of the ringwall shall be a minimum of 3/4 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. The berm shall be well-graded 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 ±1/8 in. (±3 mm) in any 10 ft (3 m) of circumference and within ±1/2 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. The berm shall be well-graded 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 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 aboveground.* 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 3. The extreme frost penetration depths in Figure 3 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 3. Uplift 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.

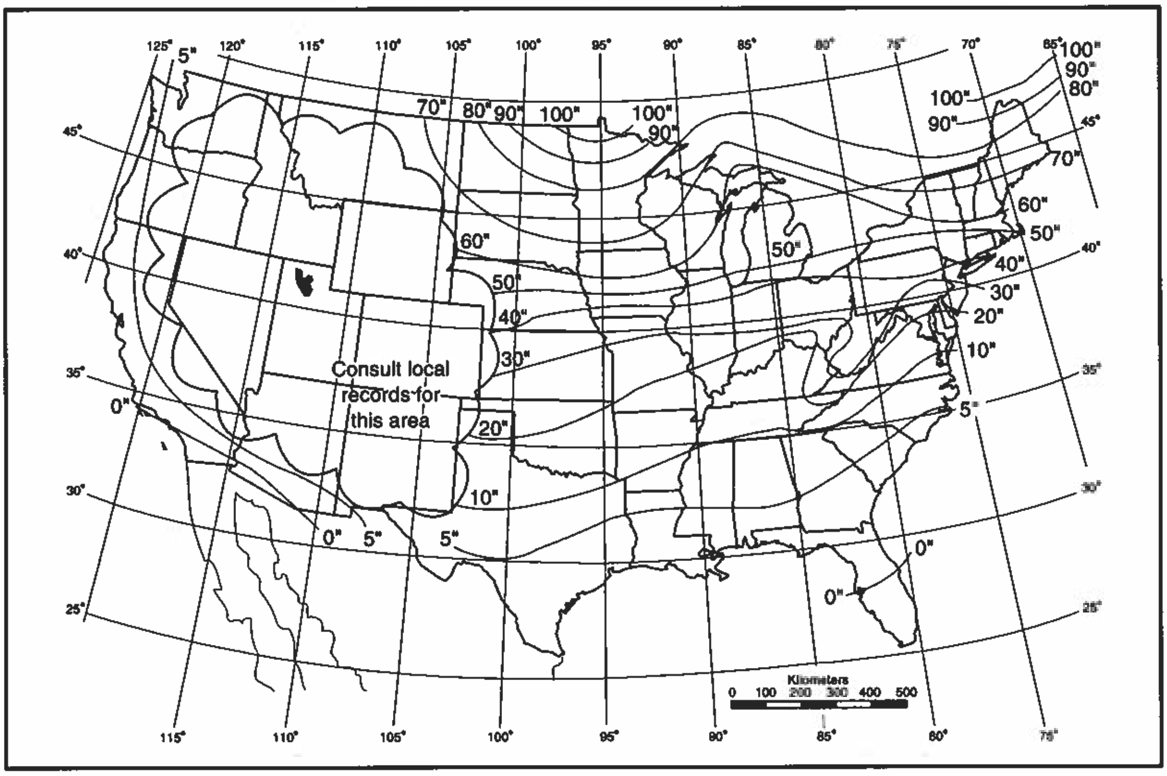

Figure 3 Extreme frost penetration—in inches (based on state average)

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

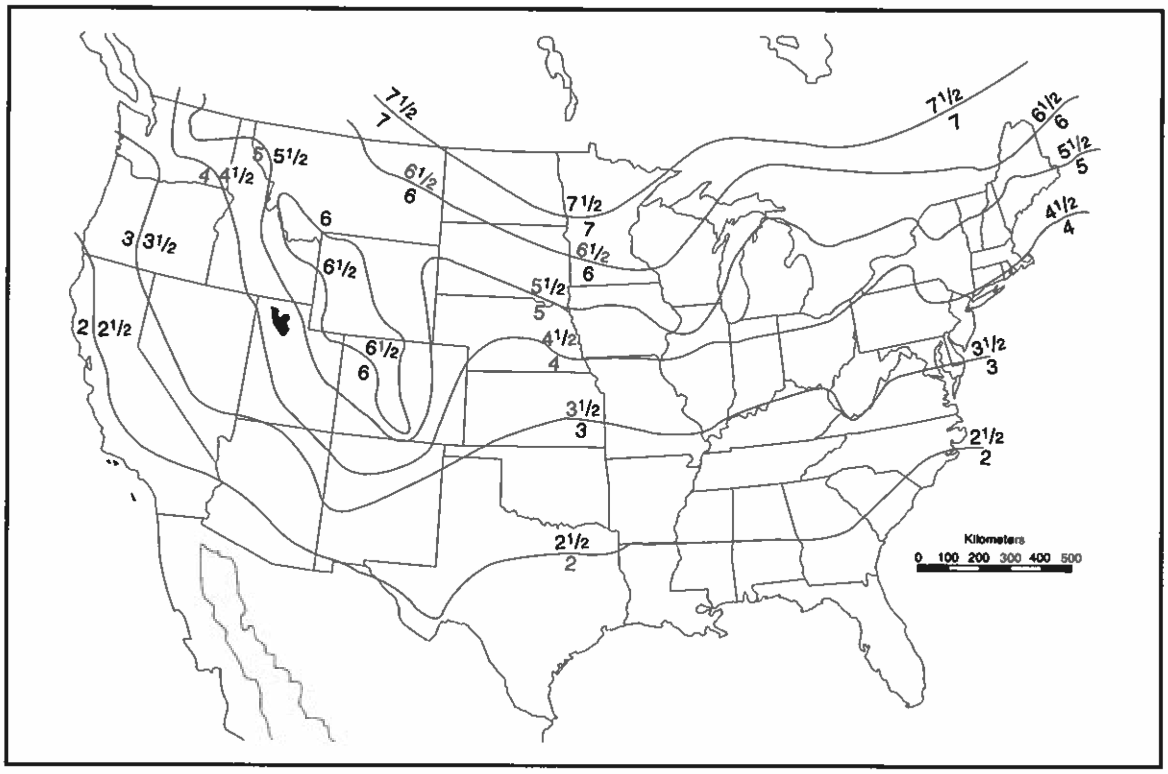

Figure 4 Recommended depth of cover (in feet above top of pipe)

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 4, unless local conditions dictate that greater or lesser cover should be used.

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 the S_1 (Sec. 13.2.3) is less than or equal to 0.04 and the period S_S (Sec. 13.2.3) 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. 13.2.7) or, when specified or required by this standard, the site-specific procedure (Sec. 13.2.8). The design seismic forces have been reduced by a factor of 1.4 and shall be used with the allowable stress design method.

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. 13.2.10 and Sec. 13.2.11.

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. 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. Tanks that rely on the inherent stability of the self-weight of the tank and contents to resist overturning forces.

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 Use Group.* The Seismic Use Group is a classification assigned to the tank based on its intended use and expected performance. The following Seismic Use Group definitions shall be used. For tanks serving multiple facilities, the facility having the highest Seismic Use Group shall be used. Seismic Use Group III shall be used unless otherwise specified.

13.2.1.1 Seismic Use Group III. Seismic Use Group III shall be used for tanks that provide direct service to facilities that are deemed essential for post-earthquake recovery and essential to the life, health, and safety of the public, including post-earthquake fire suppression.

13.2.1.2 Seismic Use Group II. Seismic Use Group II shall be used for tanks that provide direct service to facilities that are deemed important to the welfare of the public.

13.2.1.3 Seismic Use Group I. Seismic Use Group I shall be used for tanks not assigned to Seismic Use Group III or II.

13.2.2 *Seismic importance factor I_E.* The seismic importance factor I_E is based on the Seismic Use Group and shall be in accordance with Table 24.

13.2.3 *Mapped acceleration parameters.* Mapped maximum considered earthquake spectral response accelerations, 5 percent damped, at 0.2-sec period S_S and 1-sec period S_1 shall be obtained from Figures 5 through 18.

13.2.4 *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 specified as

Table 24 Seismic importance factor I_E

Seismic Use Group	Seismic Importance Factor I_E
I	1.00
II	1.25
III	1.50

one of the Site Classes in Table 25. Site Class D shall be used when the soil properties are not known in sufficient detail to determine the Site Class. A site response analysis that complies with Sec. 13.2.8 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 sec, 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 Sec. 13.2.4.1 through Sec. 13.2.4.2, and the corresponding values of F_a and F_v determined in accordance with Sec. 13.2.5.

13.2.4.1 Site classification for seismic design. The parameters used to define the Site Class are based on the upper 100 ft (30 m) of the site profile. Profiles containing distinctly different soil and rock layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft (30 m). Where some of the n layers are cohesive and others are not, k is the number of cohesive layers and m is the number of cohesionless layers. The symbol i refers to any one of the layers between 1 and n. The following parameters shall be used to classify the site:

13.2.4.1.1 Average shear wave velocity $\bar{v}_s$. The average shear wave velocity $\bar{v}_s$ shall be determined using the following equation where

$$\sum_{i=1}^{n} d_i$$

is equal to 100 ft (30 m):

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \text{(Eq 13-1)}$$

Where:

$\bar{v}_s$ = average shear wave velocity in the top 100 ft (30 m) in feet per second

d_i = thickness of layer i in feet (m)

v_{si} = shear wave velocity of layer i in feet per second

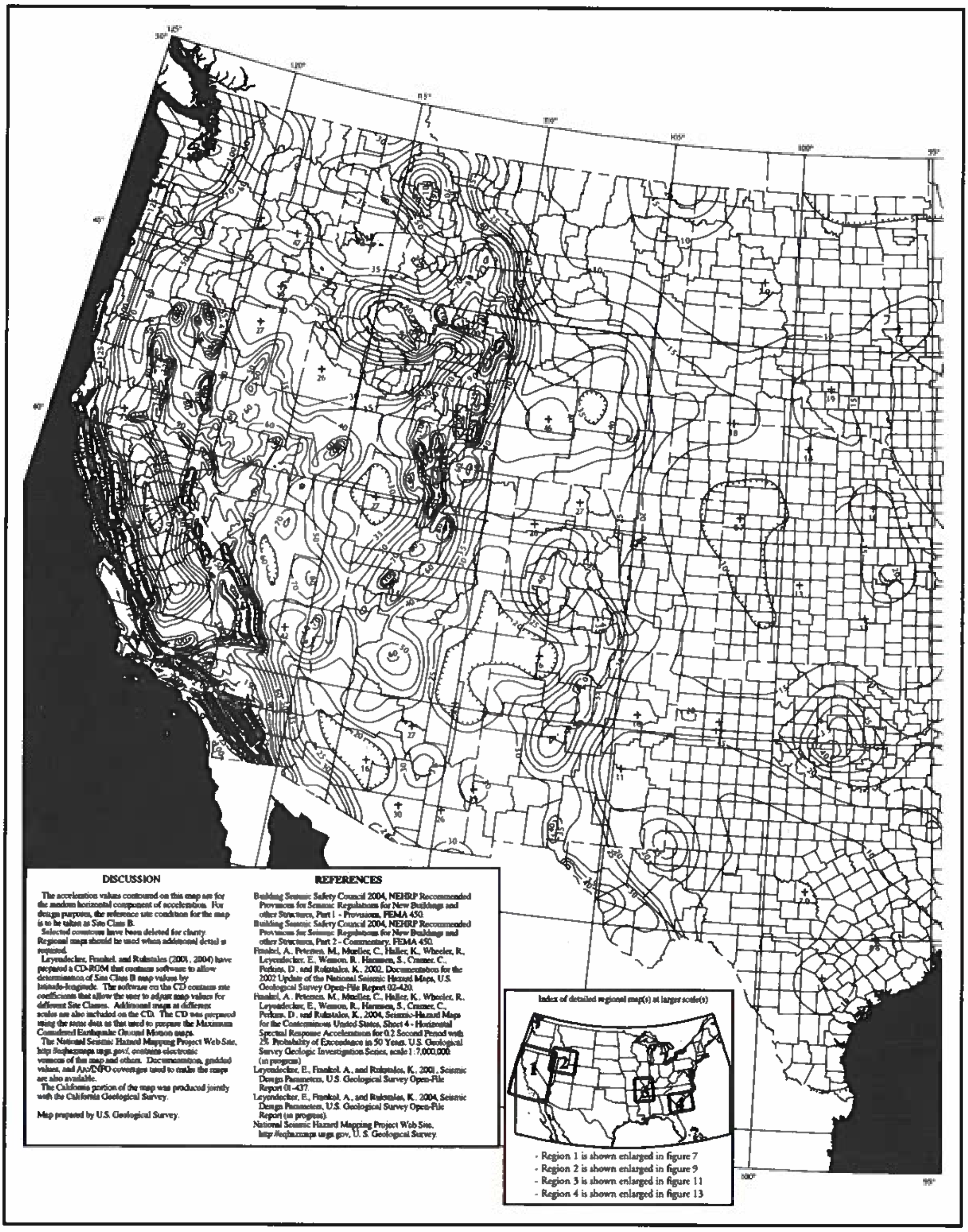

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Figure 5 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S for Site Class B for the conterminous United States (continued)

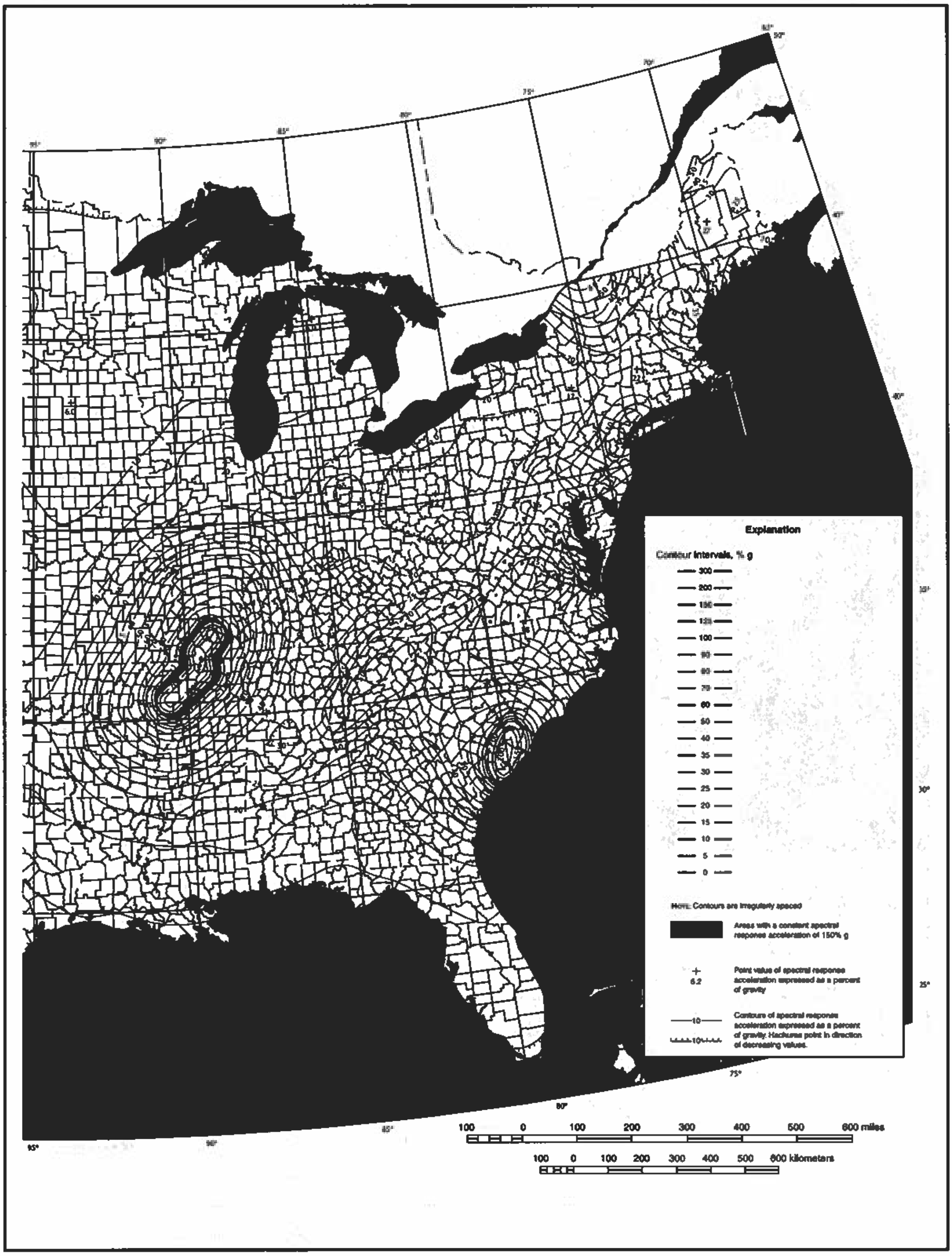

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Figure 5 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S for Site Class B for the conterminous United States

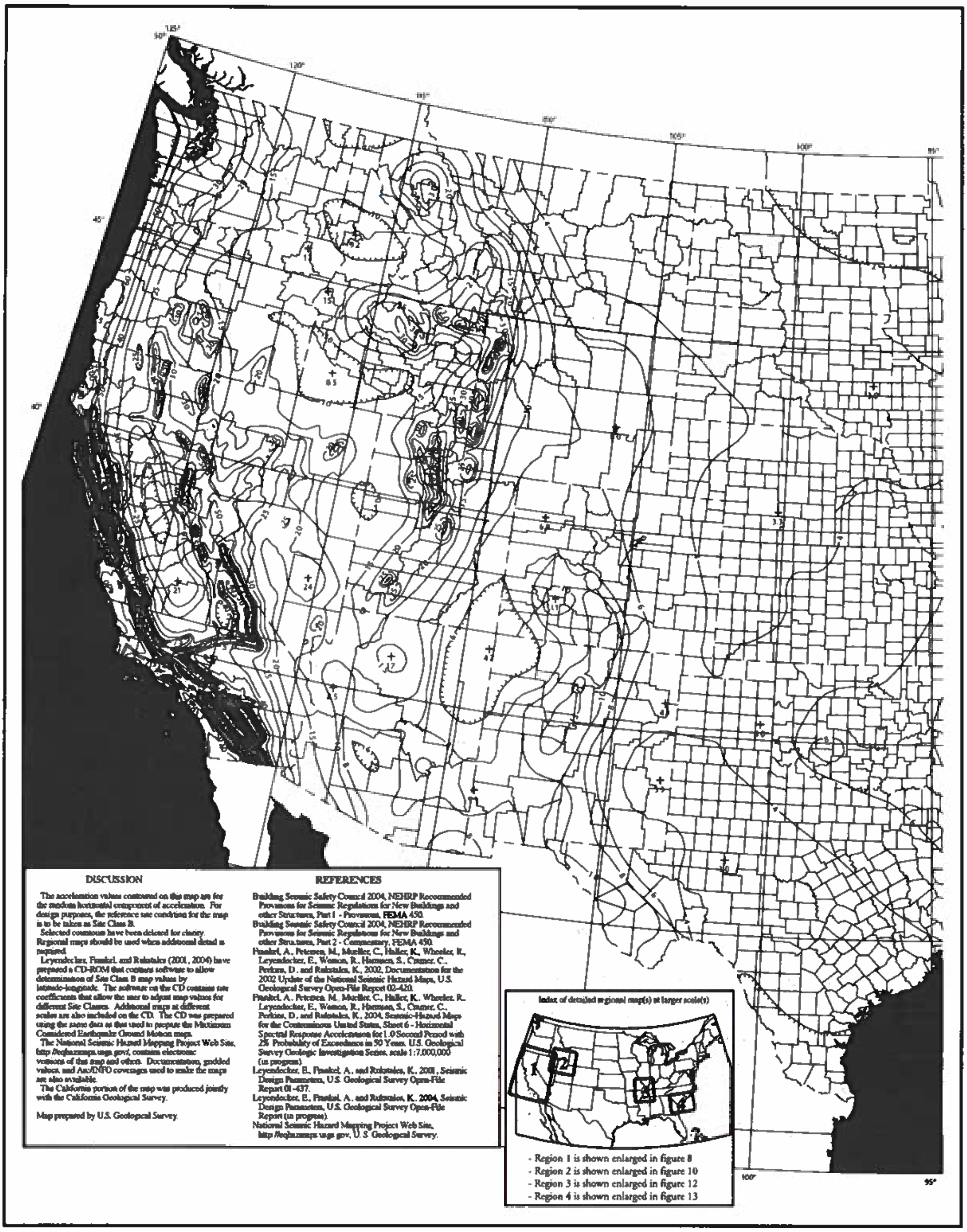

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Figure 6 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period S_1 for Site Class B for the conterminous United States (continued)

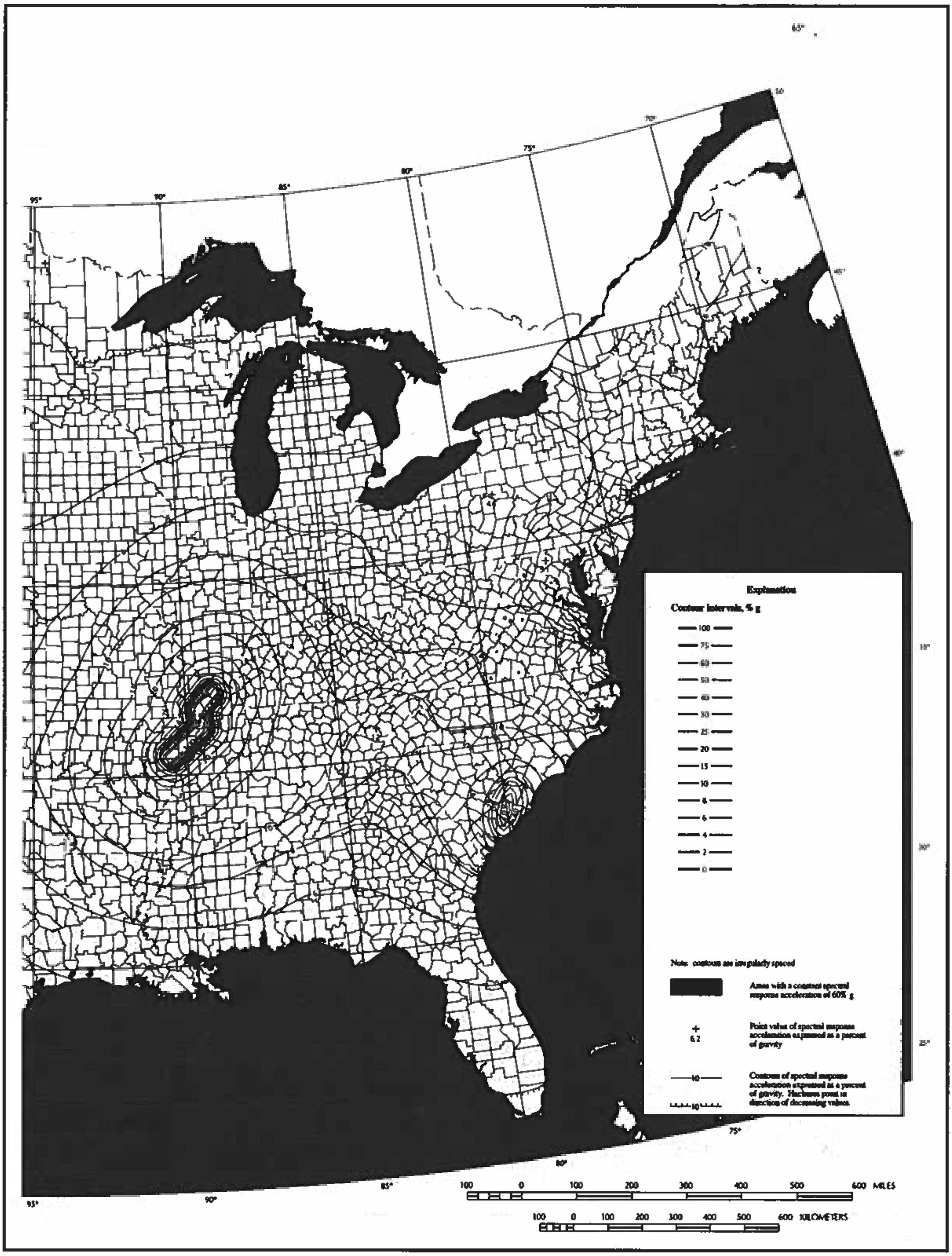

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Figure 6 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period S_1 for Site Class B for the conterminous United States

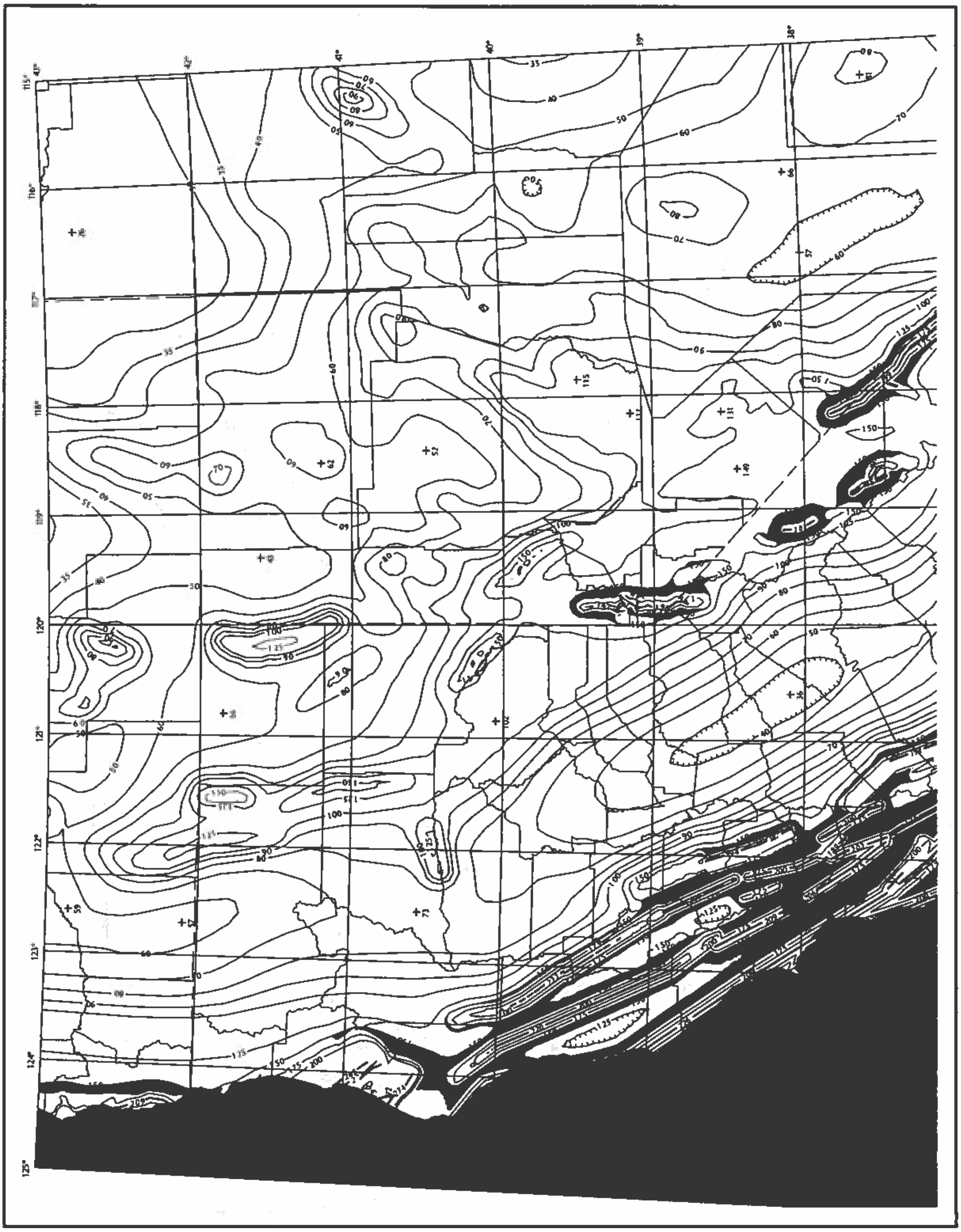

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Figure 7 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S for Site Class B for Region 1 (continued)

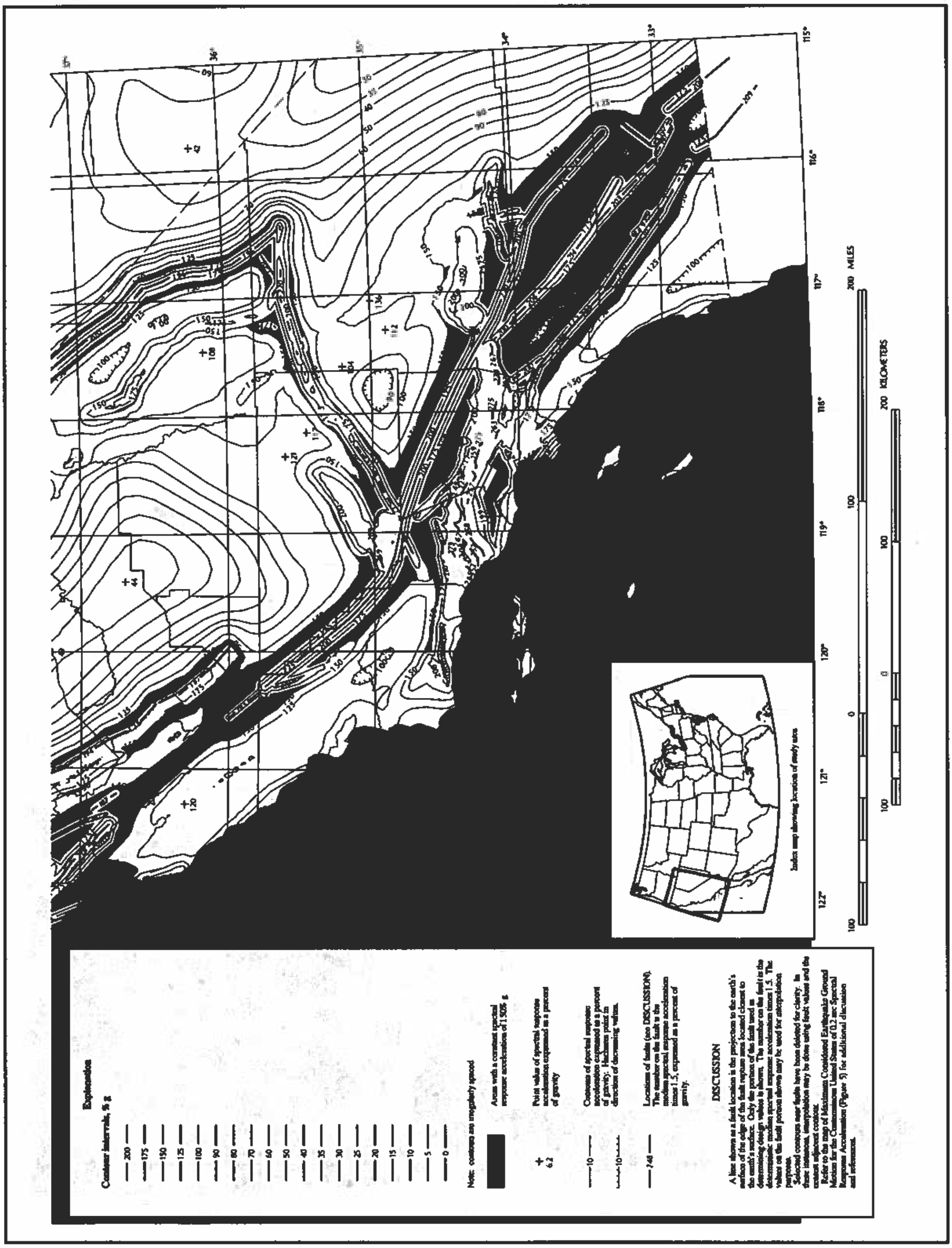

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Figure 7 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S for Site Class B for Region 1

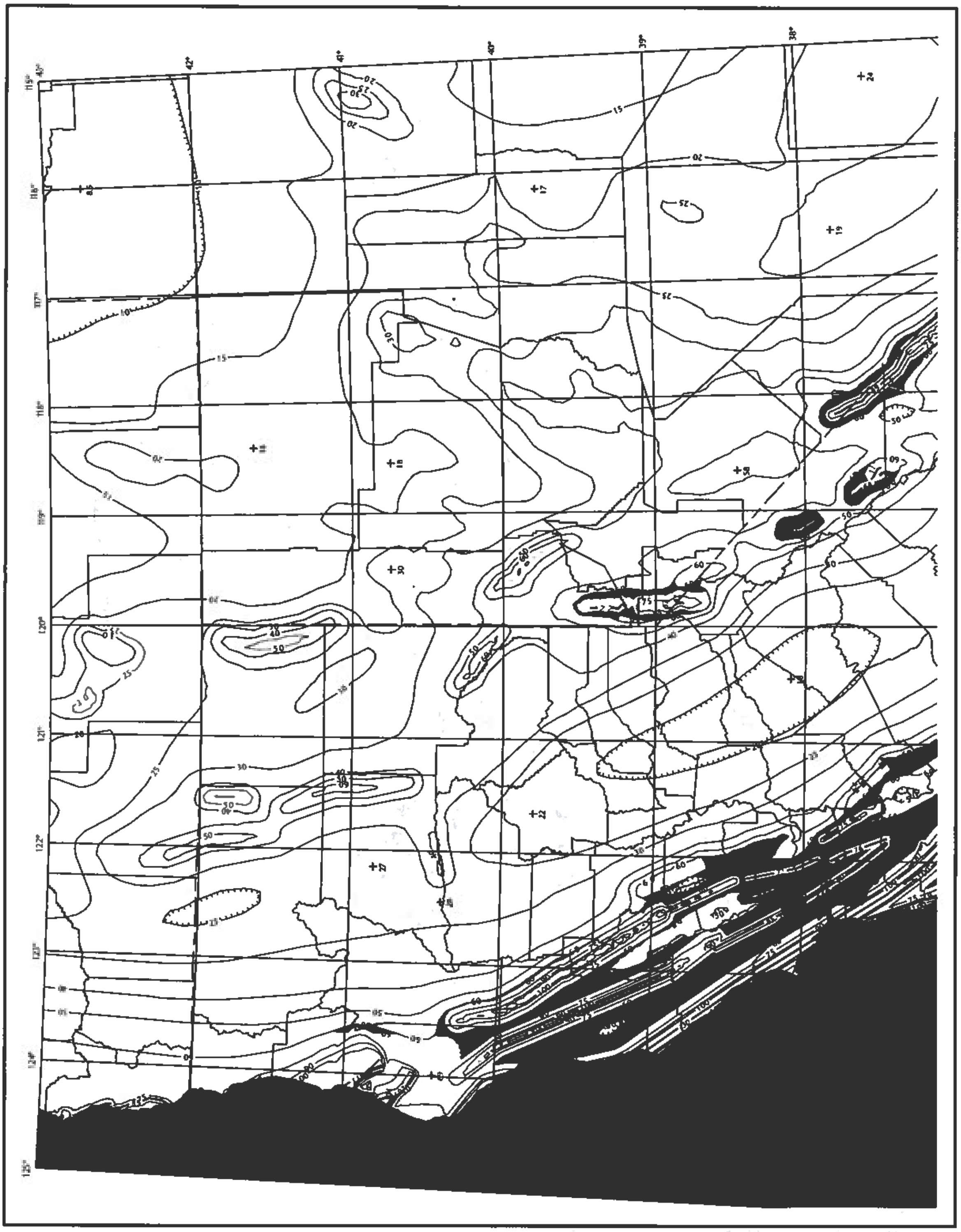

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Figure 8 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period S_1 for Site Class B for Region 1 (continued)

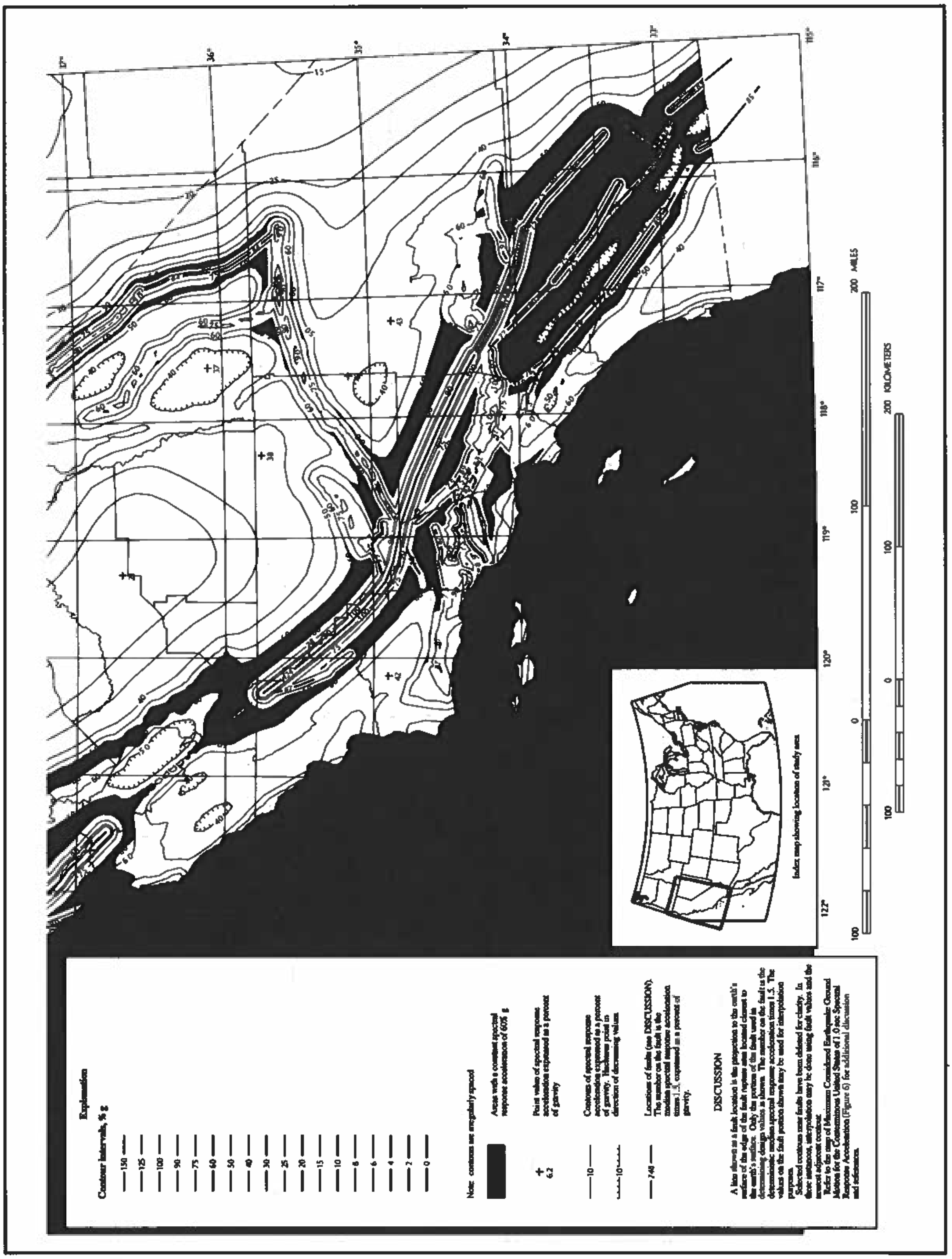

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Figure 8 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period S_1 for Site Class B for Region 1

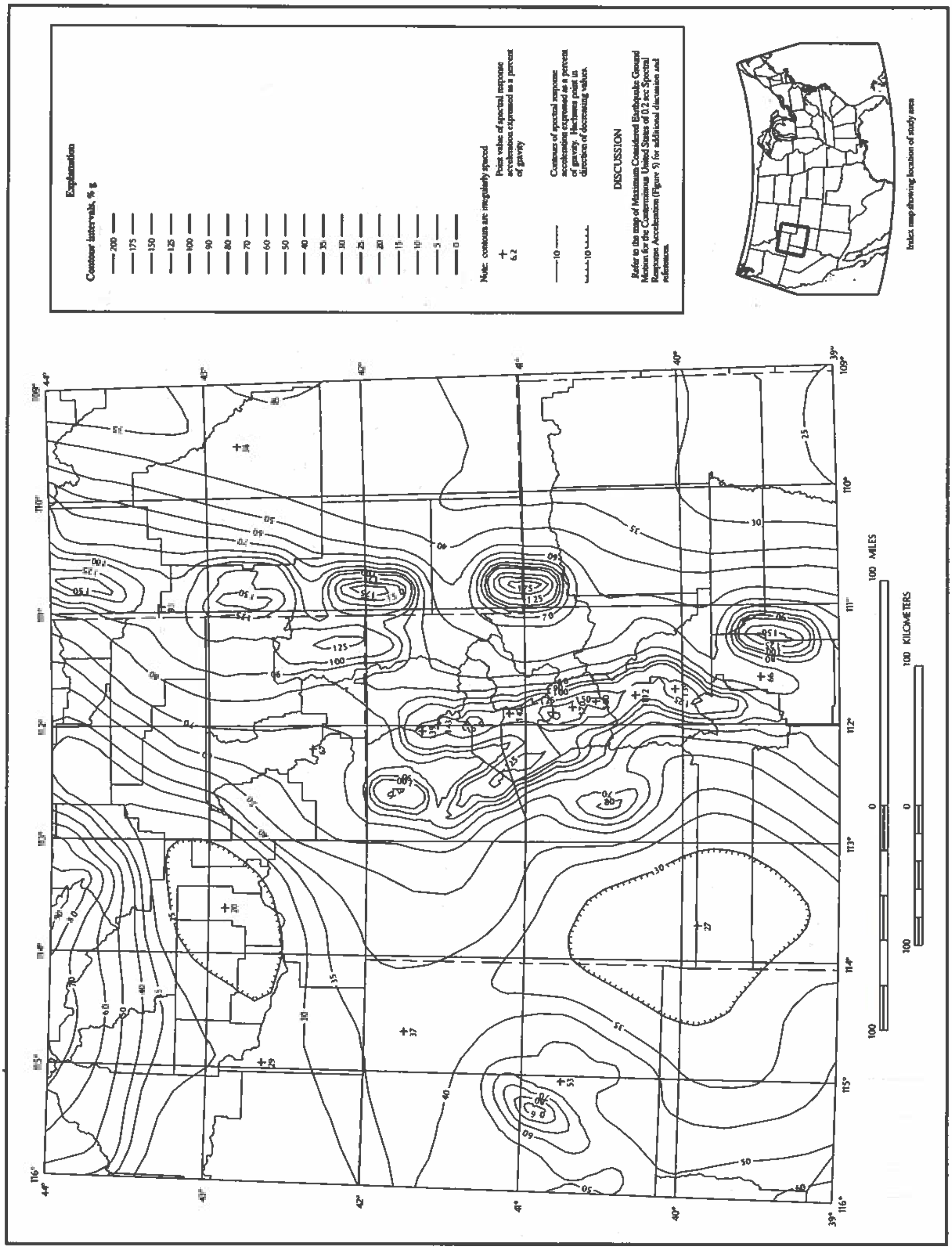

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Figure 9 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S for Site Class B for Region 2

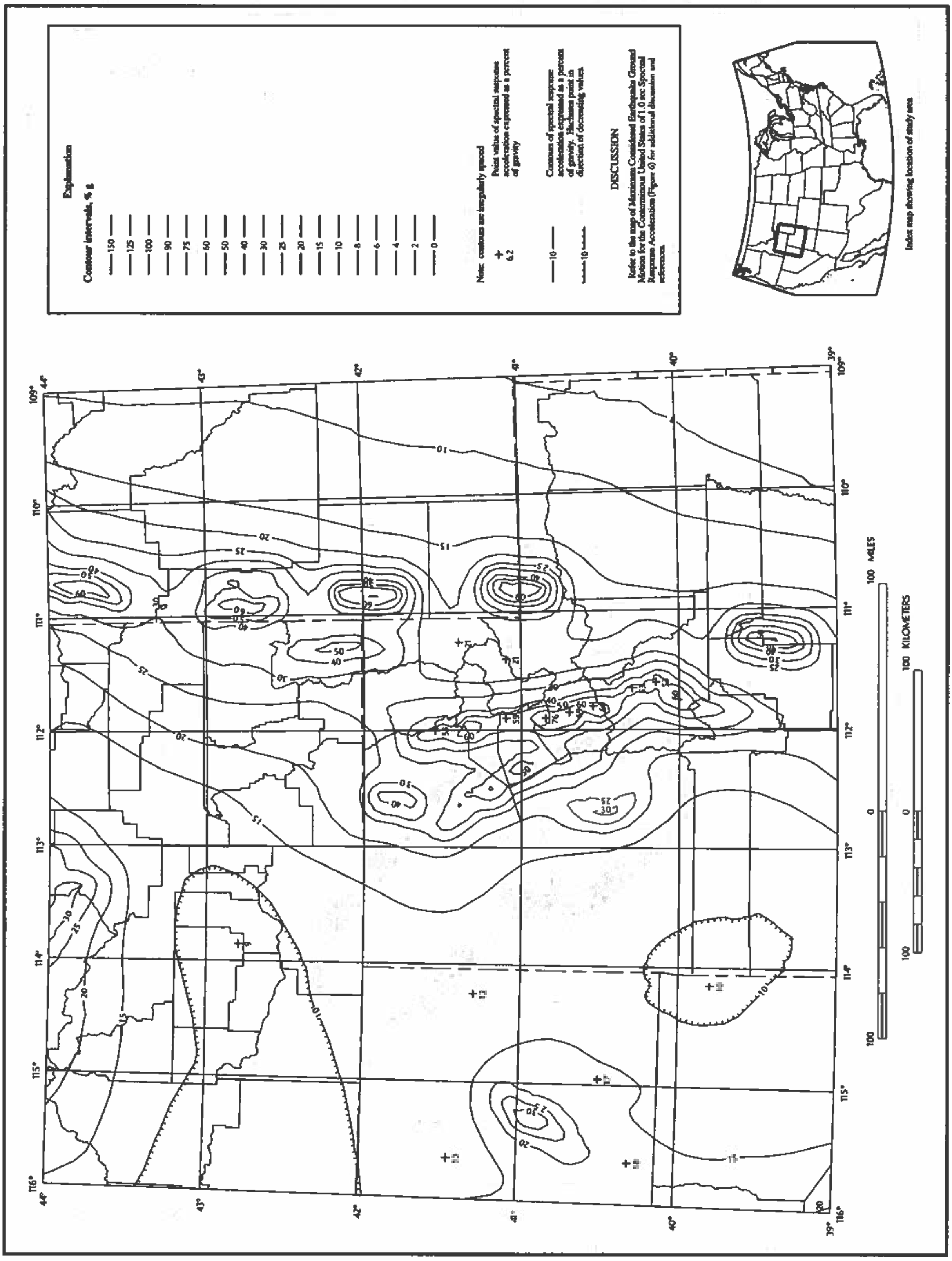

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Figure 10 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period S_1 for Site Class B for Region 2

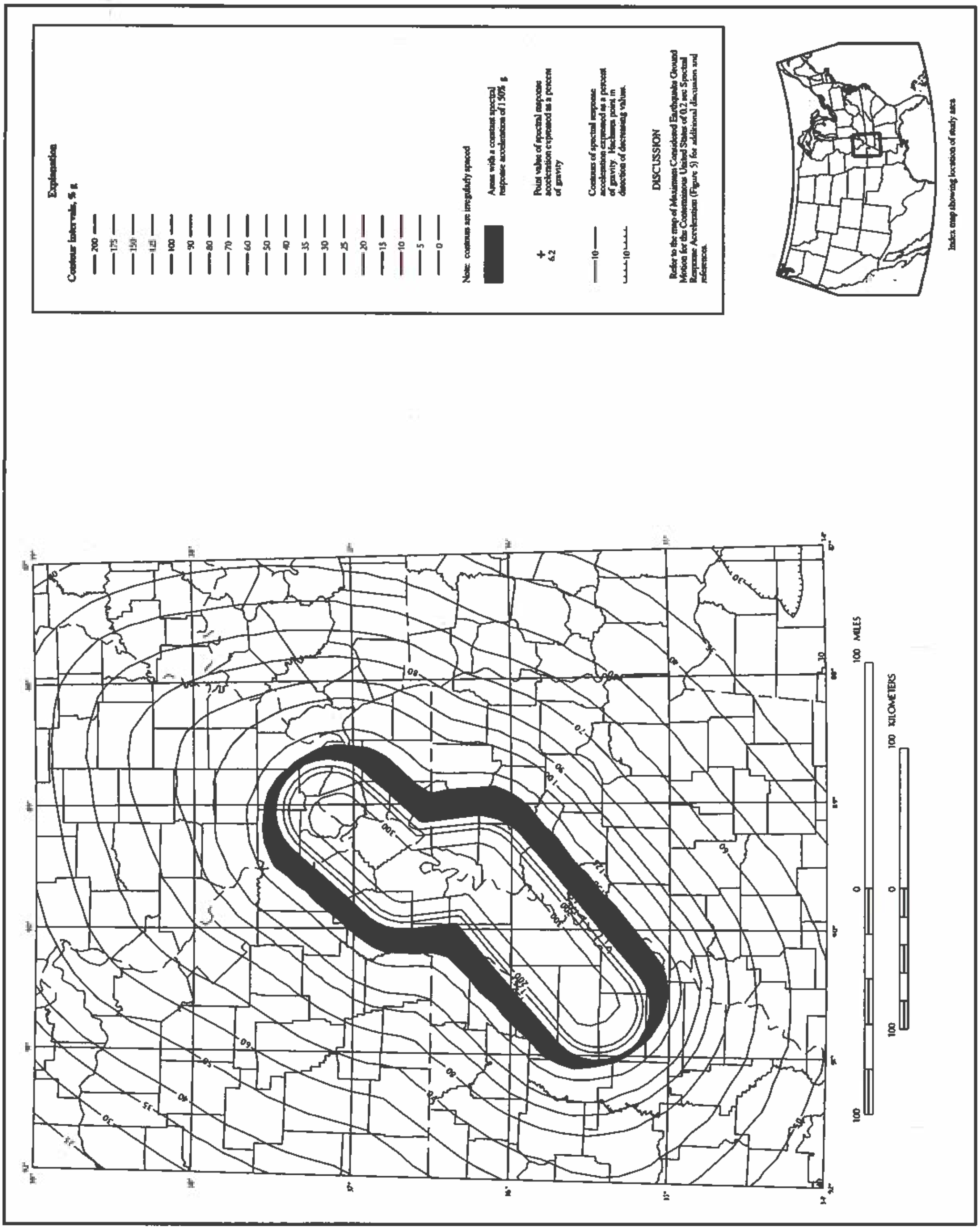

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Figure 11 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S for Site Class B for Region 3

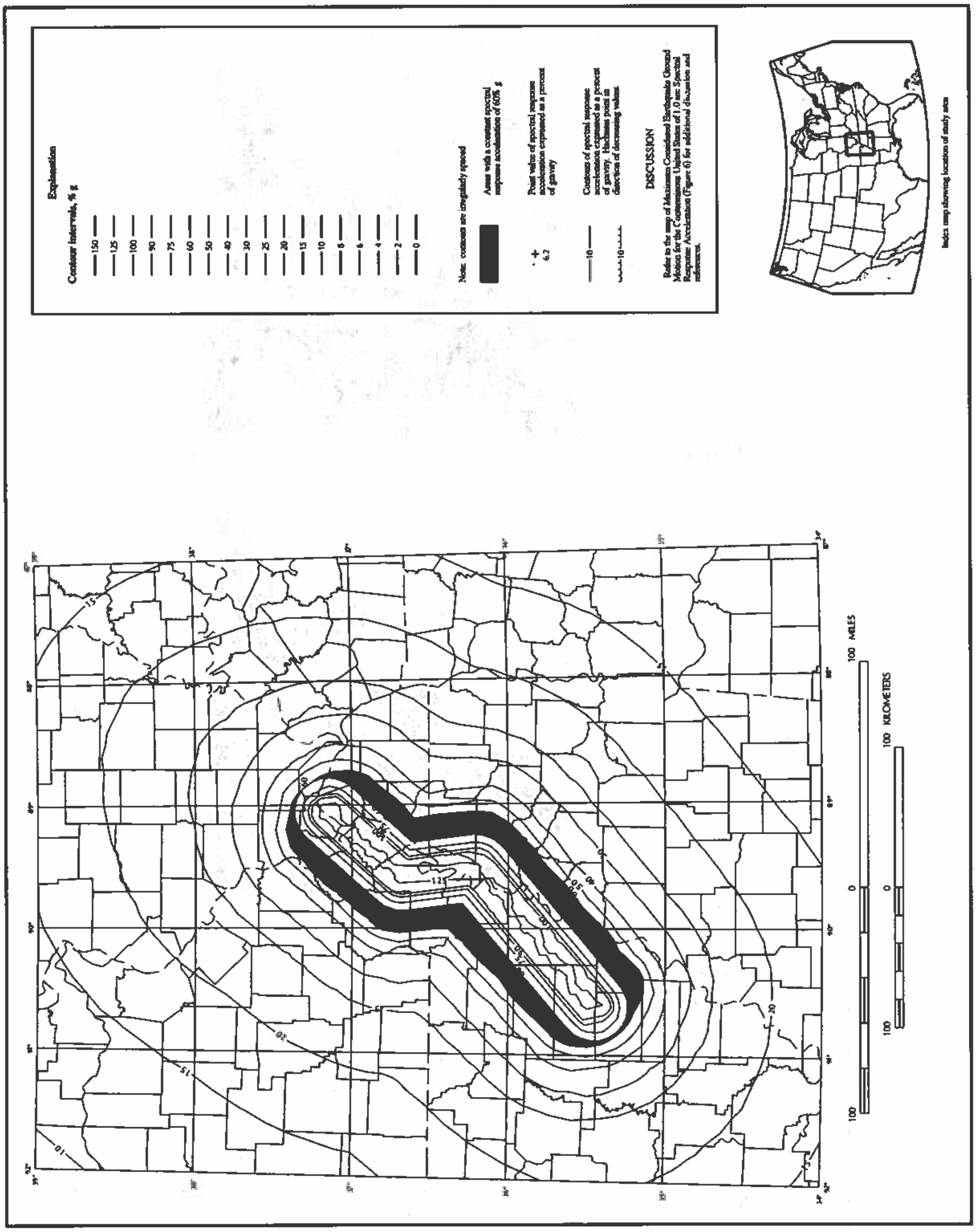

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Figure 12 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period S_1 for Site Class B for Region 3

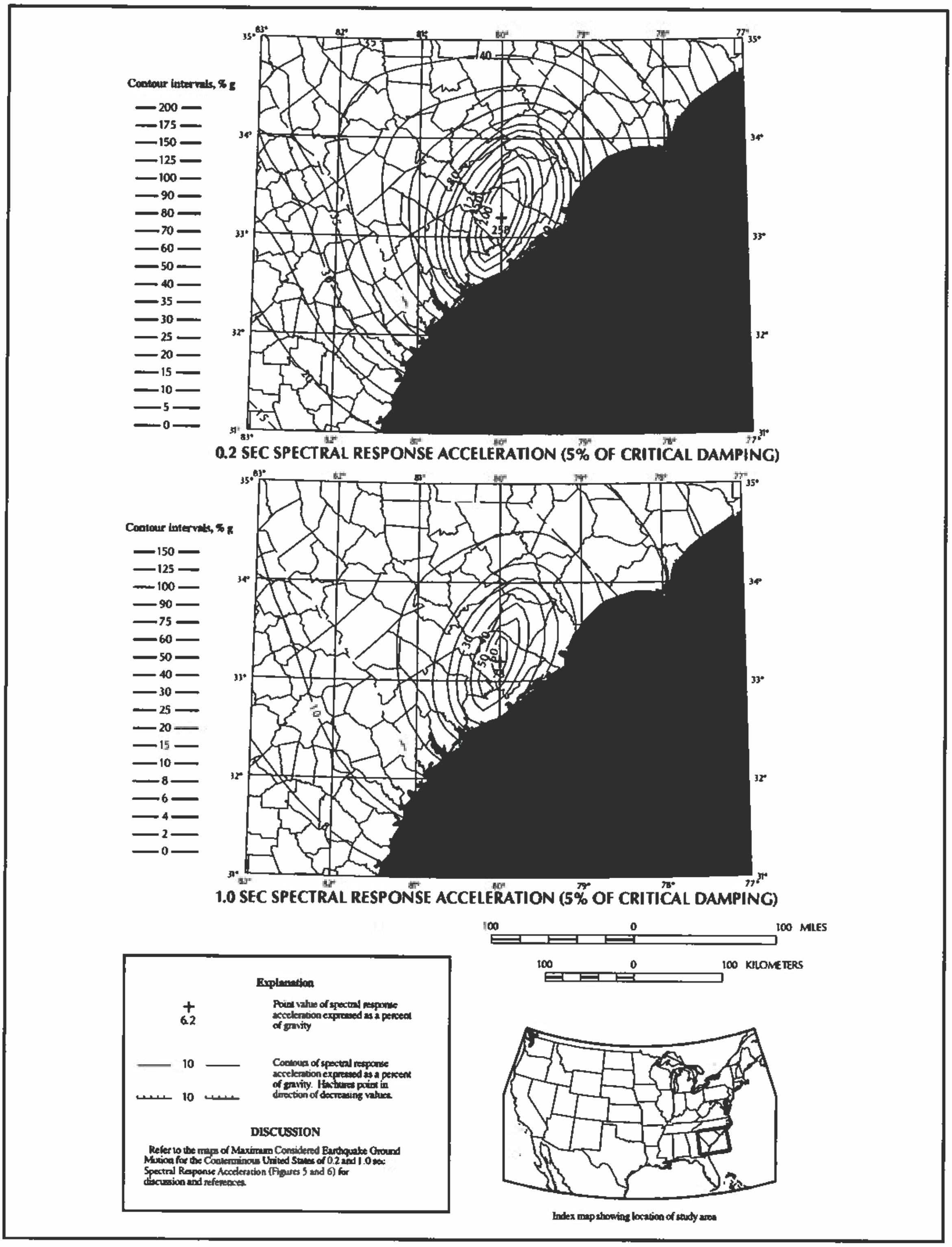

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Figure 13 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S and 1-sec period S_1 for Site Class B for Region 4

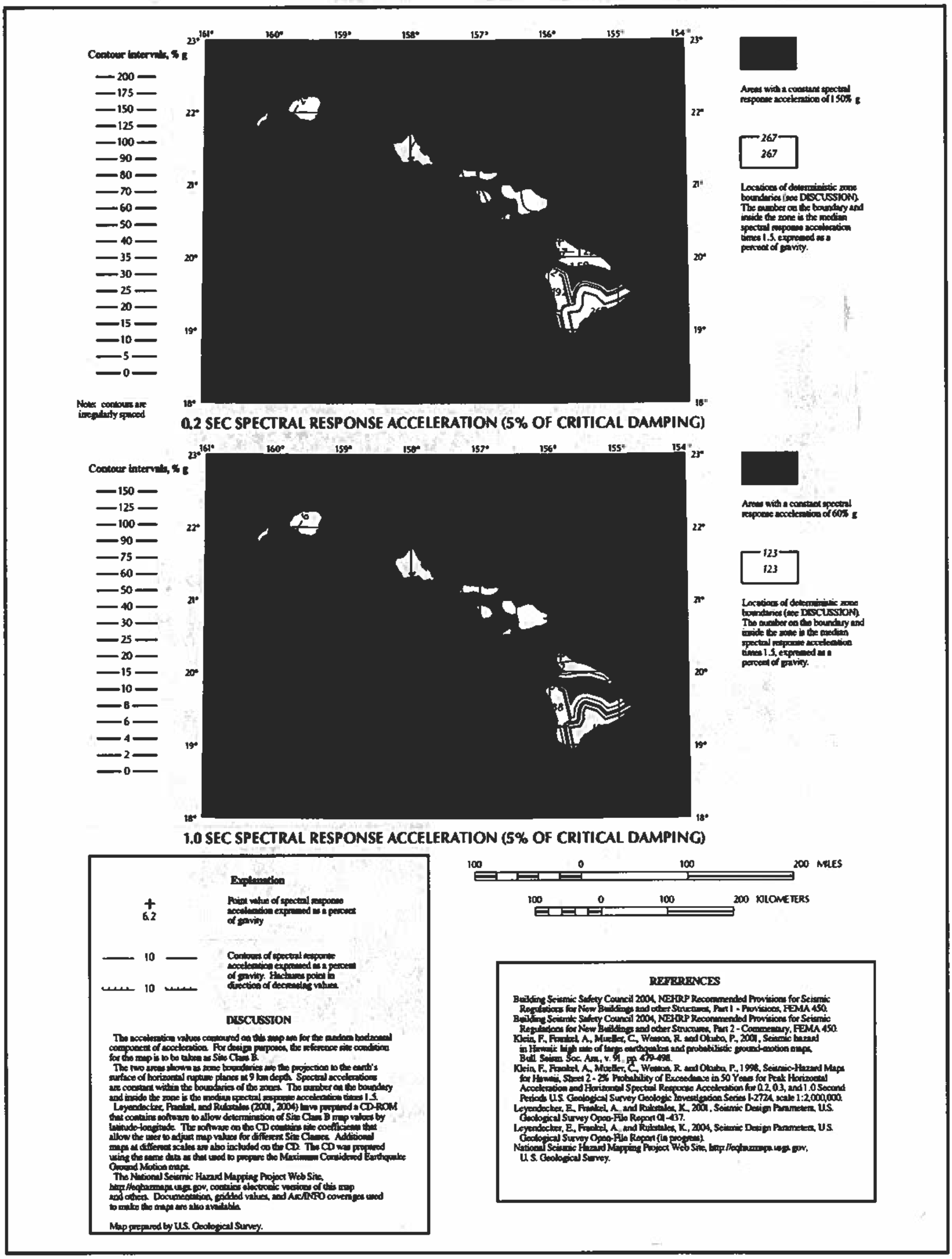

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Figure 14 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S and 1-sec period S_1 for Site Class B for Hawaii

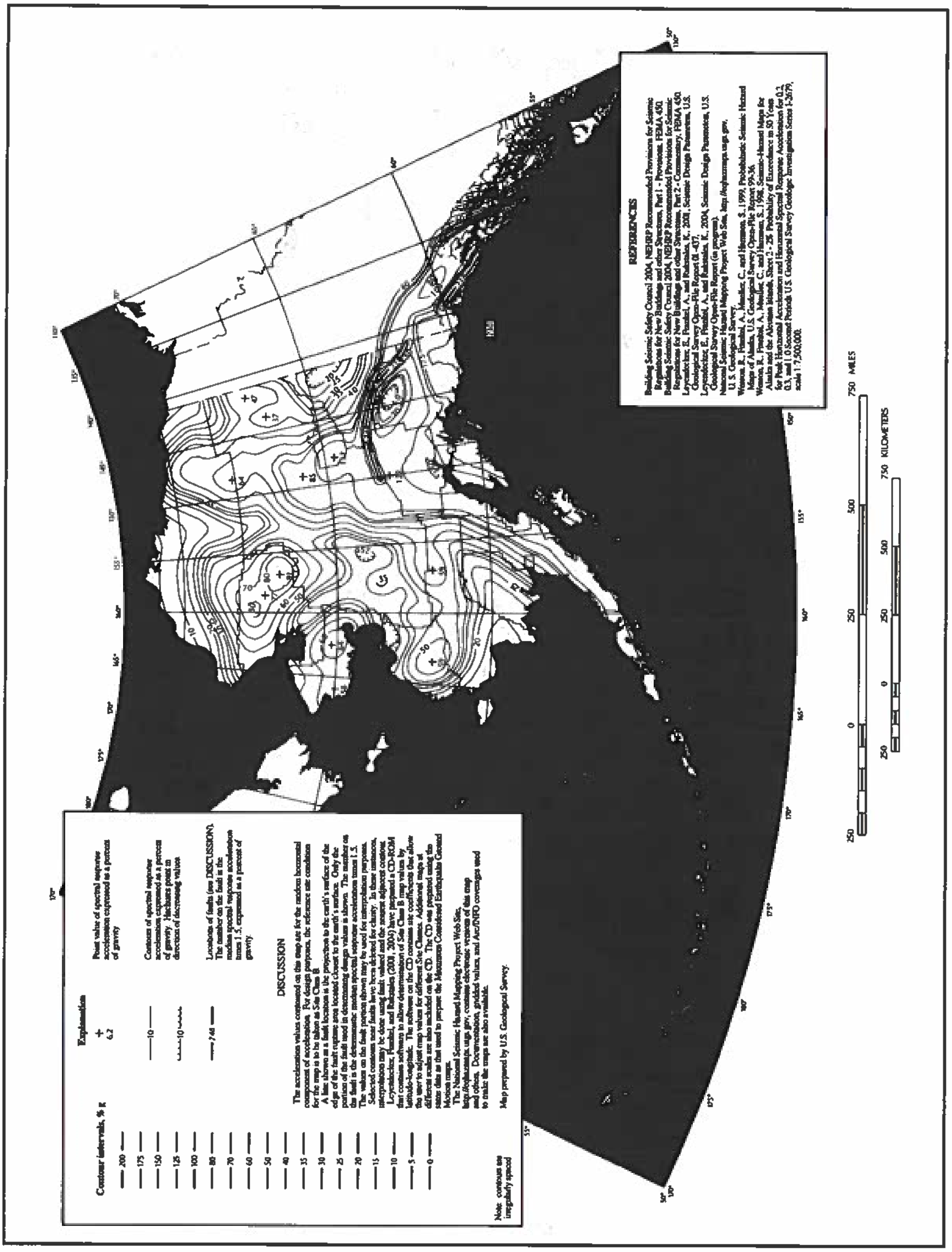

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Figure 15 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S for Site Class B for Alaska

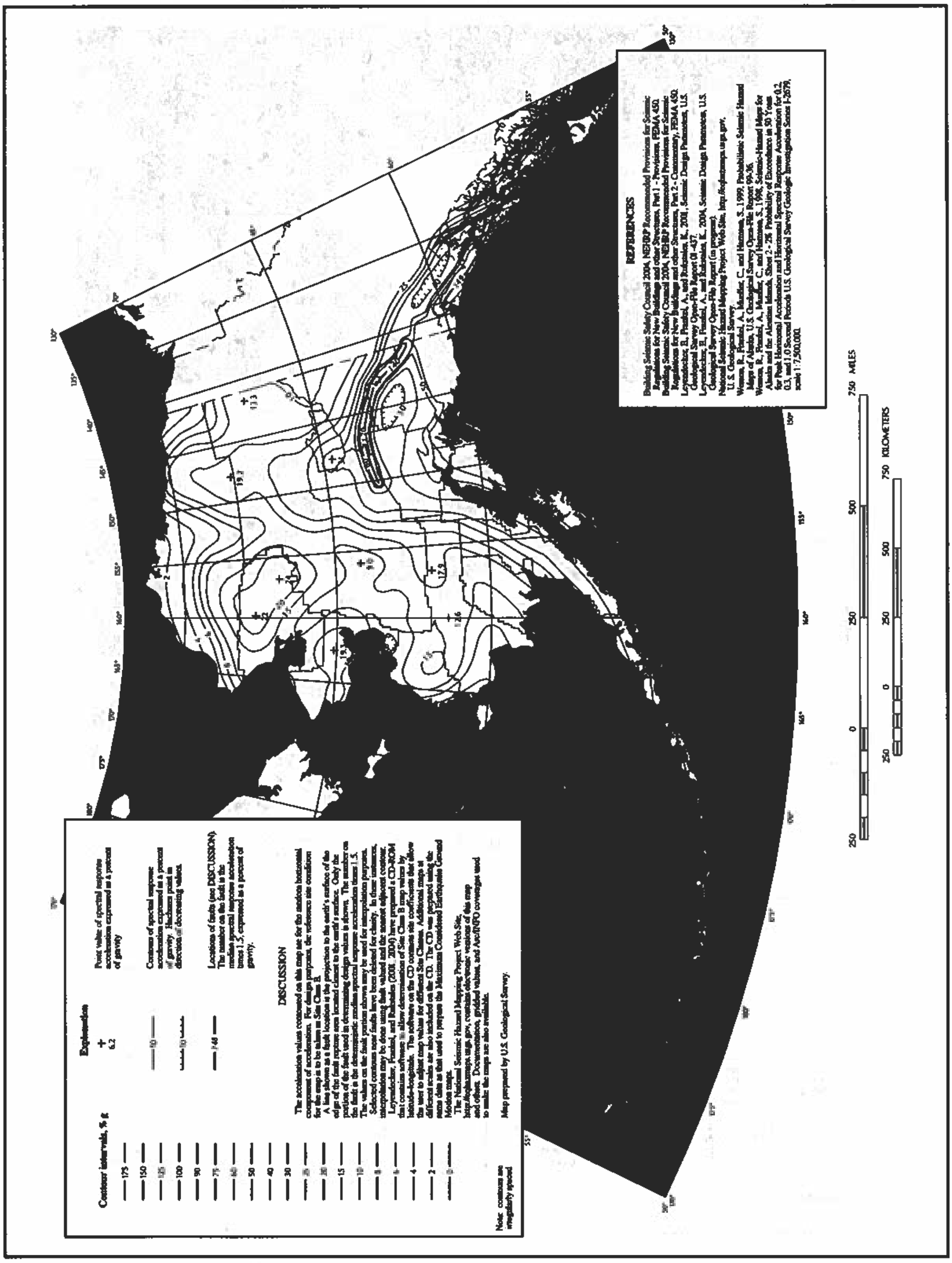

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Figure 16 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period S_1 for Site Class B for Alaska

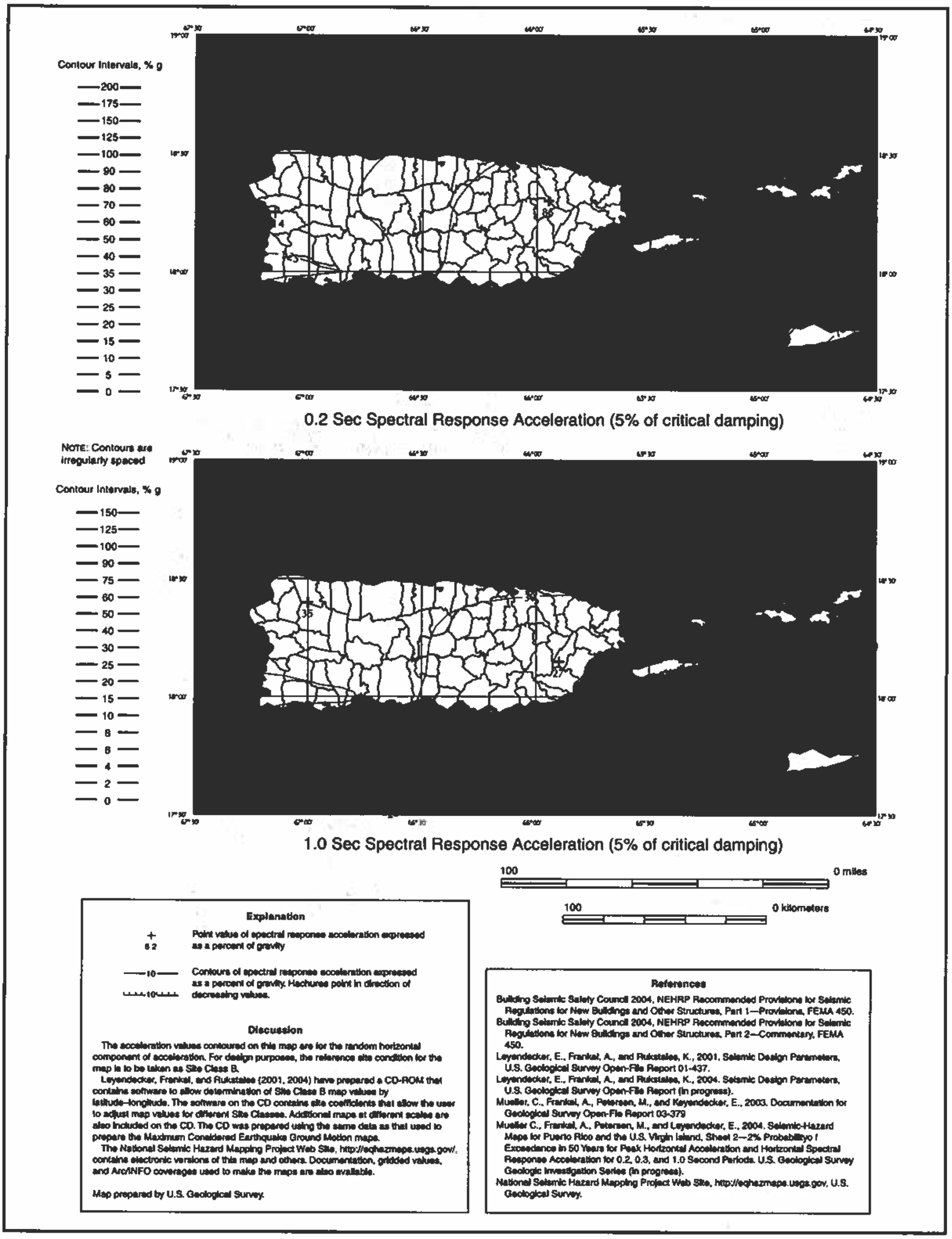

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Figure 17 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S and 1-sec period S_1 for Site Class B for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix

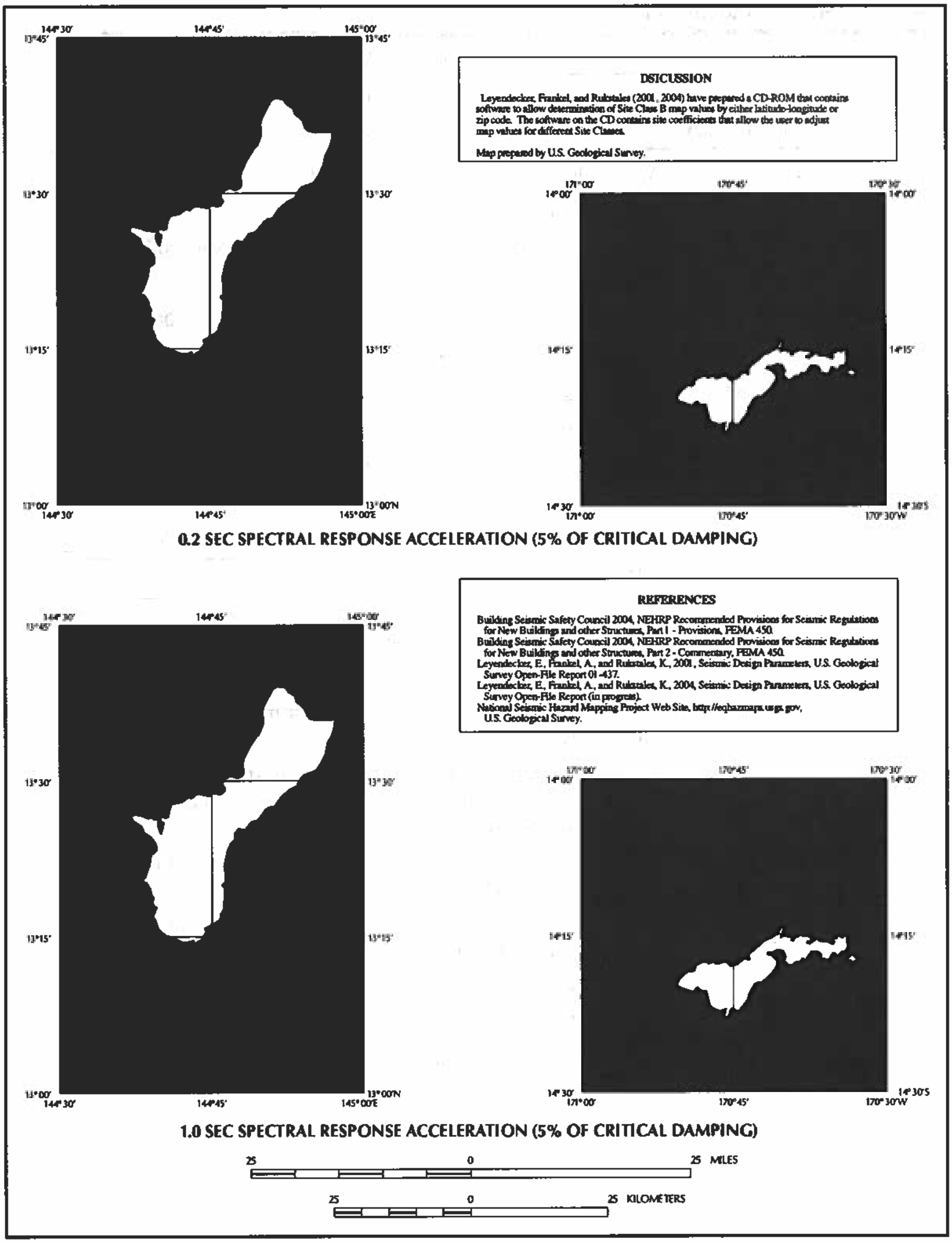

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Figure 18 Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period S_S and 1-sec period S_1 for Site Class B for Guam and Tutuila

Table 25 Site class definitions

Site Class	Soil Profile Name	Average Properties in Top 100 ft (30 m) Shear Wave Velocity $\bar{v}_s$ (ft/s)	Standard Penetration Resistance $\overline{N}$ or $\overline{N}_{ch}$	Undrained Shear Strength $\bar{s}_u$ (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$ ($\bar{v}_s > 1{,}500$ m/s)	Not applicable	Not applicable
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$ (760 m/s $< \bar{v}_s \leq 1{,}500$ m/s)	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$ (370 m/s $< \bar{v}_s \leq 760$ m/s)	$\overline{N}$ or $\overline{N}_{ch} > 50$	$\bar{s}_u > 2{,}000$ ($\bar{s}_u > 100$ kPa)
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$ (180 m/s $\leq \bar{v}_s \leq 370$ m/s)	$15 \leq \overline{N}$ or $\overline{N}_{ch} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$ (50 kPa $\leq \bar{s}_u \leq 100$ kPa)
E	Soft soil profile	$\bar{v}_s < 600$ ($\bar{v}_s < 180$ m/s)	$\overline{N}$ or $\overline{N}_{ch} < 15$	$\bar{s}_u < 1{,}000$ ($\bar{s}_u < 50$ kPa)
E	or	Any profile with more than 10 ft (3 m) of soil having all of the following characteristics: 1. Plasticity index $PI > 20$ 2. Moisture content $w \geq 40$ percent 3. Undrained shear strength $\bar{s}_u < 500$ psf (24 kPa)		
F*	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils 2. Peats and/or highly organic clays (more than 10 ft (3 m) of peat and/or highly organic clay) 3. Very high plasticity clays (more than 25 ft (7.6 m) of soil thickness with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays (more than 120 ft (36.5 m) of soil thickness)		

*Site-specific evaluation and procedure (Sec. 13.2.8) are required.

13.2.4.1.2 Average standard penetration resistance $\bar{N}$ or $\bar{N}_{ch}$. The average field standard penetration resistance $\bar{N}$ for cohesionless soil, cohesive soil, and rock layers shall be determined using Eq 13-2 where

$$\sum_{i=1}^{n} d_i$$

is equal to 100 ft (30 m).

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \text{(Eq 13-2)}$$

The average standard penetration resistance for cohesionless soil layers only shall be determined using Eq 13-3 where

$$\sum_{i=1}^{m} d_i$$

is equal to d_s.

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \qquad \text{(Eq 13-3)}$$

Where:

$\bar{N}$ or $\bar{N}_{ch}$ = average standard penetration in the top 100 ft (30 m) in blows per foot

N_i = standard penetration resistance of layer i in blows per foot. N_i shall be determined in accordance with ASTM D1586 and measured directly in the field without corrections. N_i shall not be taken greater than 100 blows/ft (328 blows/m). Where refusal is met for a rock layer, N_i shall be taken as 100 blows/ft (328 blows/m).

d_s = total thickness of cohesionless soil layers in the top 100 ft (30 m) in feet

The other symbols have been previously defined in this section.

13.2.4.1.3 Average undrained shear strength $\bar{s}_u$. The average undrained shear strength $\bar{s}_u$ shall be determined using

$$\sum_{i=1}^{k} d_i = d_c$$

in the following equation:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \qquad \text{(Eq 13-4)}$$

Where:

$\bar{s}_u$ = average undrained shear strength in the top 100 ft (30 m) in pounds per square foot

d_c = total thickness of cohesive soil layers in the top 100 ft (30 m) in feet

s_{ui} = undrained shear strength of layer i in pounds per square foot. The undrained shear strength shall be determined in accordance with ASTM D2166 or D2850 and shall not be taken greater than 5,000 psf (250 kPa).

The other symbols have been previously defined in this section.

13.2.4.2 Procedure for classifying a site. The following procedure shall be used when classifying a site:

13.2.4.2.1 Check for the four characteristics of Site Class F (Table 25) requiring site-specific evaluation. If the site has any of these characteristics, classify the site as Site Class F and conduct a site-specific evaluation (Sec. 13.2.8.1).

13.2.4.2.2 Check for the existence of a total thickness of soft clay greater than 10 ft (3 m). If the layer has all three of the characteristics of soft clay ($\bar{s}_u < 500$, $w \geq 40$ percent, and $PI > 20$), classify the site as Site Class E.

13.2.4.2.3 Classify the site as Site Class E, D, or C based on one of the following parameters and Table 25:

1. Average shear wave velocity $\bar{v}_s$ in the top 100 ft (30 m).

2. Average standard penetration resistance $\overline{N}$ in the top 100 ft (30 m).

3. Average standard penetration resistance $\overline{N_{ch}}$ for cohesionless soil layers ($PI \leq 20$) in the top 100 ft (30 m) and average undrained shear strength $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft (30 m). If the average undrained shear strength $\bar{s}_u$ is used and the $\overline{N_{ch}}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils.

13.2.4.2.4 Assignment of Site Class B shall be based on the shear wave velocity for rock. For competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the site shall be assigned to Site Class C. Site Class B shall not be used where there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

13.2.4.2.5 Assignment of Site Class A shall be supported by either shear wave velocity measurements on-site or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft (30 m), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$. Site Class A shall not be used where there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

13.2.5 *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-sec and 1-sec periods, respectively, for site classes other than B. Site coefficients F_a and F_v shall be in accordance with Tables 26 and 27, respectively.

13.2.6 *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 28.

Table 26 Short-period site coefficient F_a

	Mapped Spectral Response Acceleration at 5 Percent Damping and 0.2-Sec 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. 13.2.8) are required.

Table 27 Long-period site coefficient F_v

	Mapped Spectral Response Acceleration at 5 Percent Damping and 1-Sec Period*				
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	†	†	†	†	†

* Use straight-line interpolation for intermediate values of S_1.
† Site-specific evaluation and procedure (Sec. 13.2.8) are required.

Table 28 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 alternate fluid–structure interaction procedure is used (Sec. 13.2.10).

13.2.7 *Design response spectra—general procedure.*

13.2.7.1 General. The general procedure is based on the mapped maximum considered earthquake spectral response accelerations from Figures 5 through 18 for an event with a 2 percent probability of exceedance within a 50-year period.

13.2.7.2 Maximum considered earthquake spectral response acceleration. Mapped maximum considered earthquake spectral response accelerations, 5 percent damped, at 0.2-sec period S_S and 1-sec period S_1 for Site Class B from Figures 5 through 18 shall be adjusted for site-class effects, as shown in Eq 13-5 and Eq 13-6.

$$S_{MS} = F_a S_S \qquad \text{(Eq 13-5)}$$

$$S_{M1} = F_v S_1 \qquad \text{(Eq 13-6)}$$

Where:

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

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

S_S = mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-sec period for Site Class B from Figures 5 through 18, stated as a multiple (decimal) of g

S_1 = mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec period for Site Class B from Figures 6 through 18, stated as a multiple (decimal) of g

F_a = short-period site coefficient from Table 26

F_v = long-period site coefficient from Table 27

g = acceleration due to gravity

13.2.7.3 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-sec period S_{DS} and 1-sec period S_{D1}.

$$S_{DS} = US_{MS} \quad \text{(Eq 13-7)}$$

$$S_{D1} = US_{M1} \quad \text{(Eq 13-8)}$$

Where:

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

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

U = scaling factor = ⅔ to scale the maximum considered earthquake spectral response acceleration to the design earthquake spectral response acceleration

The other symbols have been previously defined in this section.

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

$$\text{For } 0 \le T_i \le T_S\text{: } S_{ai} = S_{DS} \quad \text{(Eq 13-9)}$$

$$\text{For } T_S < T_i \le T_L\text{: } S_{ai} = \frac{S_{D1}}{T_i} \le S_{DS} \quad \text{(Eq 13-10)}$$

$$\text{For } T_i > T_L\text{: } S_{ai} = \frac{T_L S_{D1}}{T_i^2} \quad \text{(Eq 3-11)}$$

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, shown in Figure 19

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

The other symbols have been previously defined in this section.

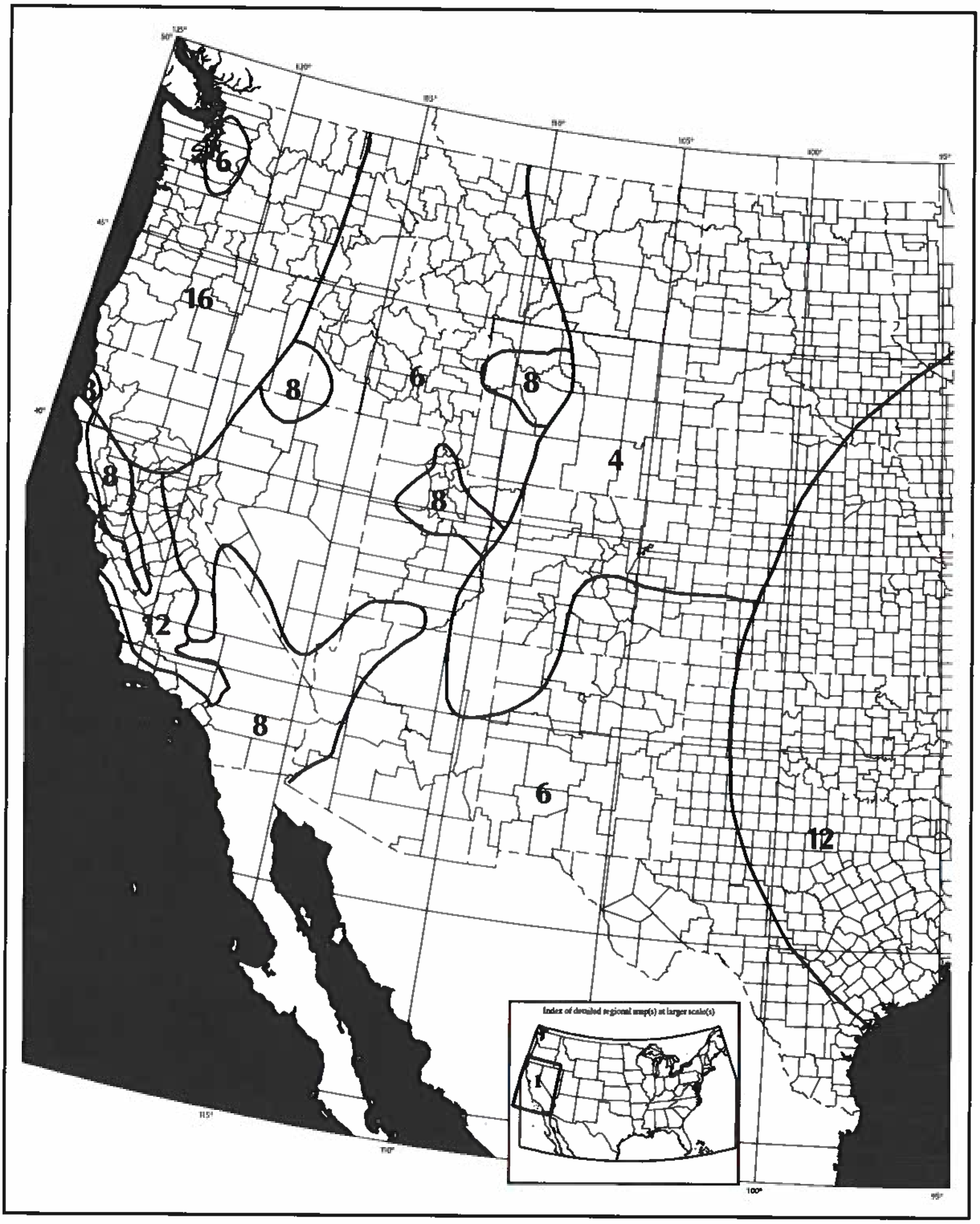

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Figure 19 Region-dependent transition period for longer-period ground motion T_L (continued)

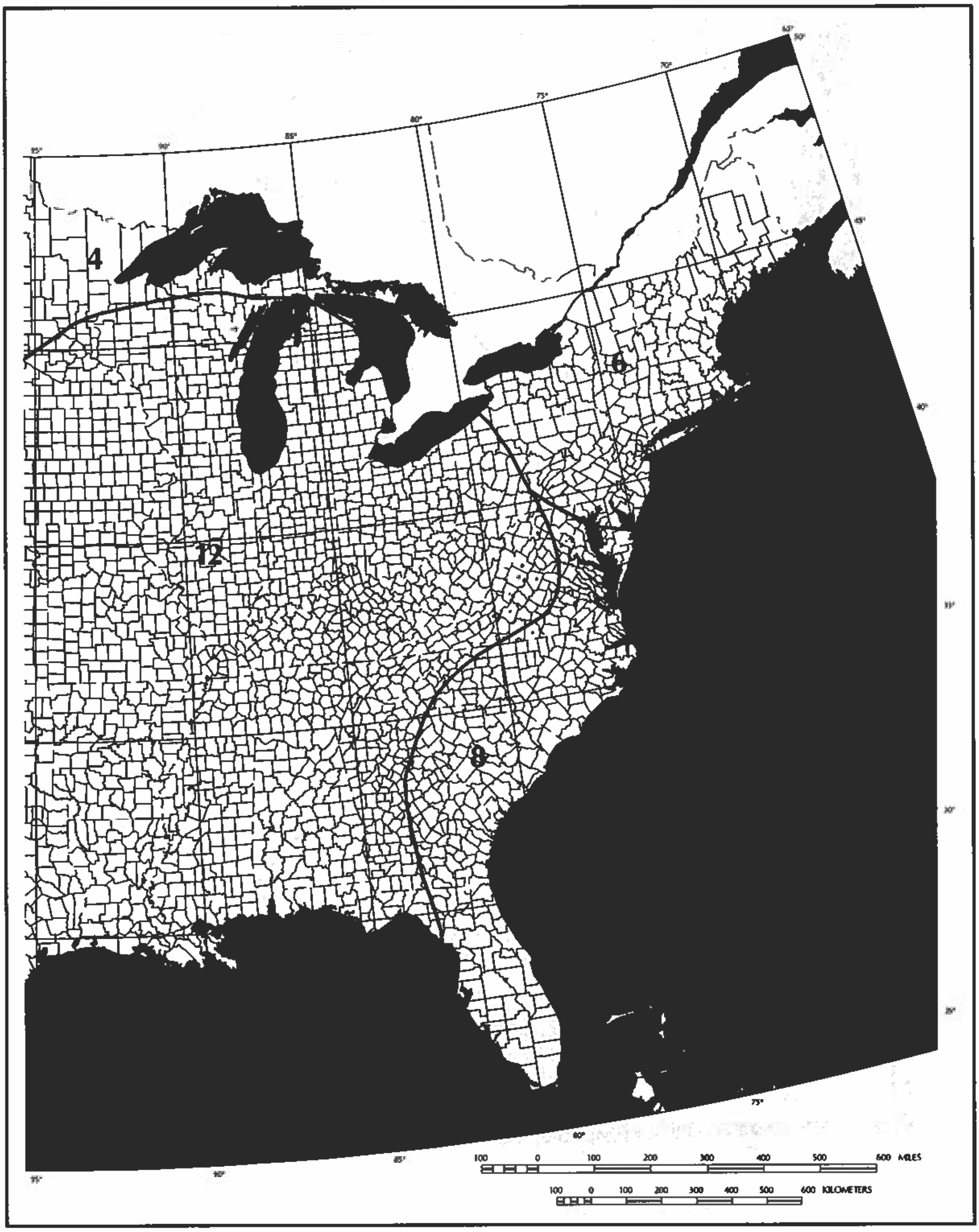

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Figure 19 Region-dependent transition period for longer-period ground motion T_L (continued)

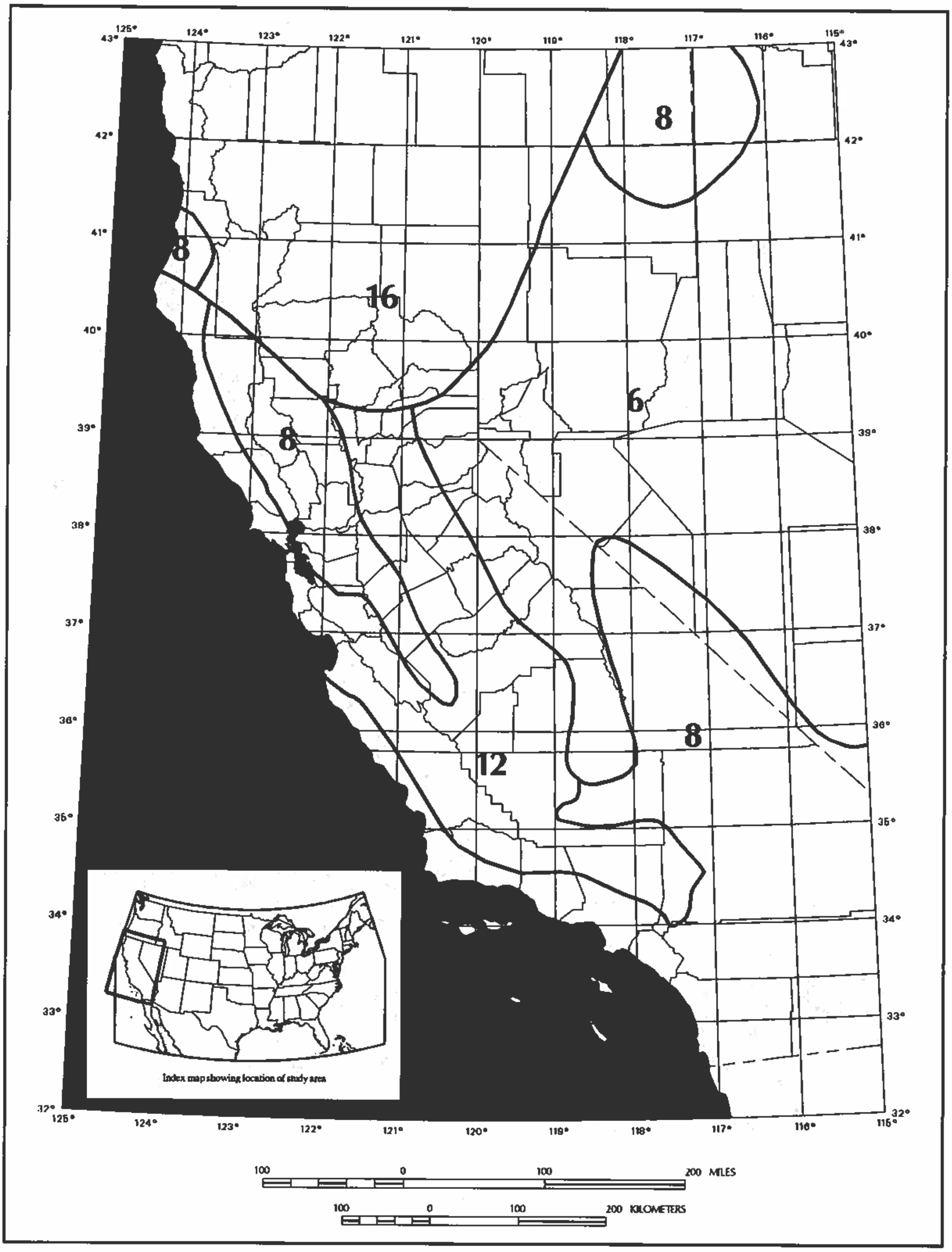

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Figure 19 Region-dependent transition period for longer-period ground motion T_L (continued)

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Figure 19 Region-dependent transition period for longer-period ground motion T_L (continued)

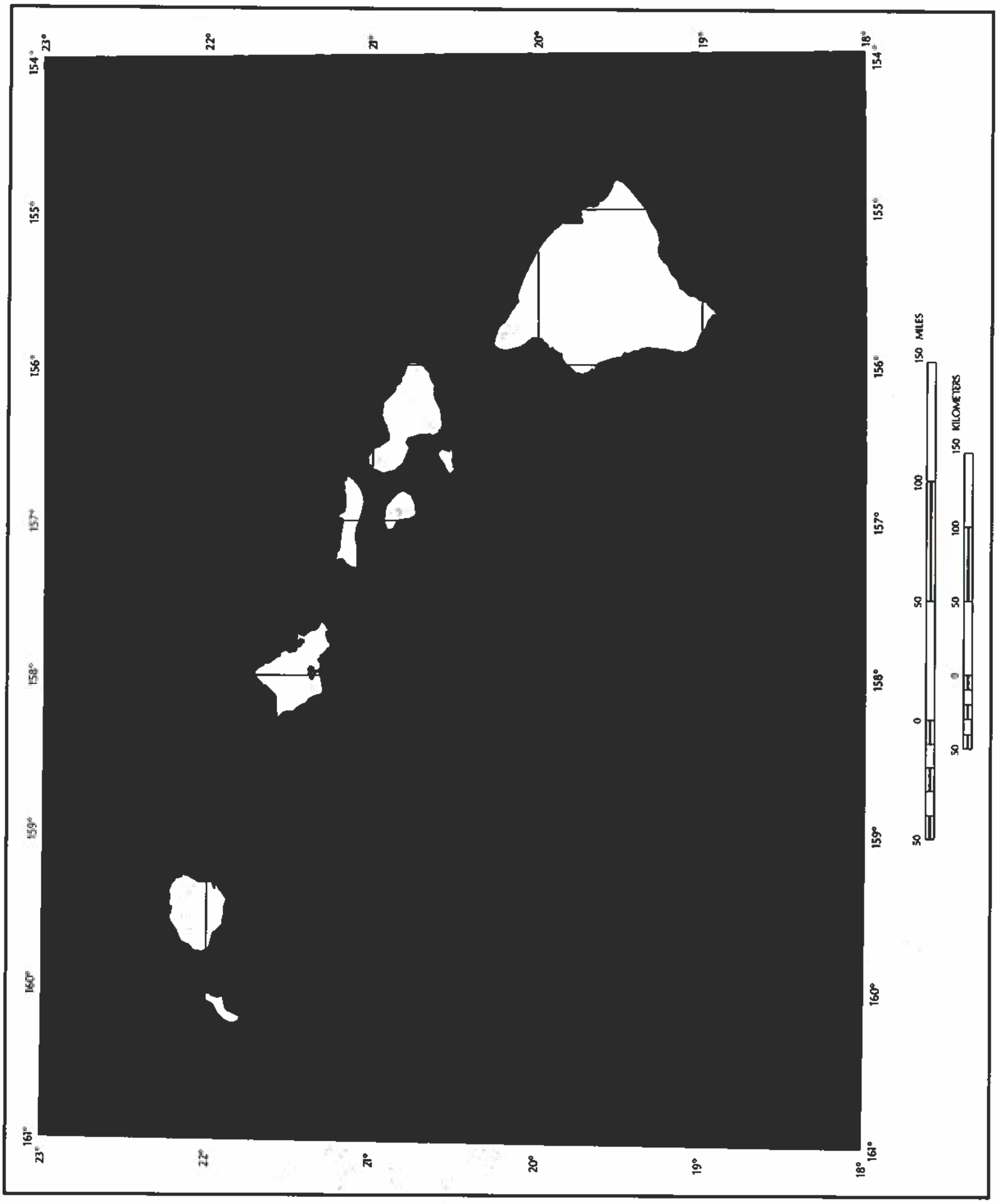

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Figure 19 Region-dependent transition period for longer-period ground motion T_L (continued)

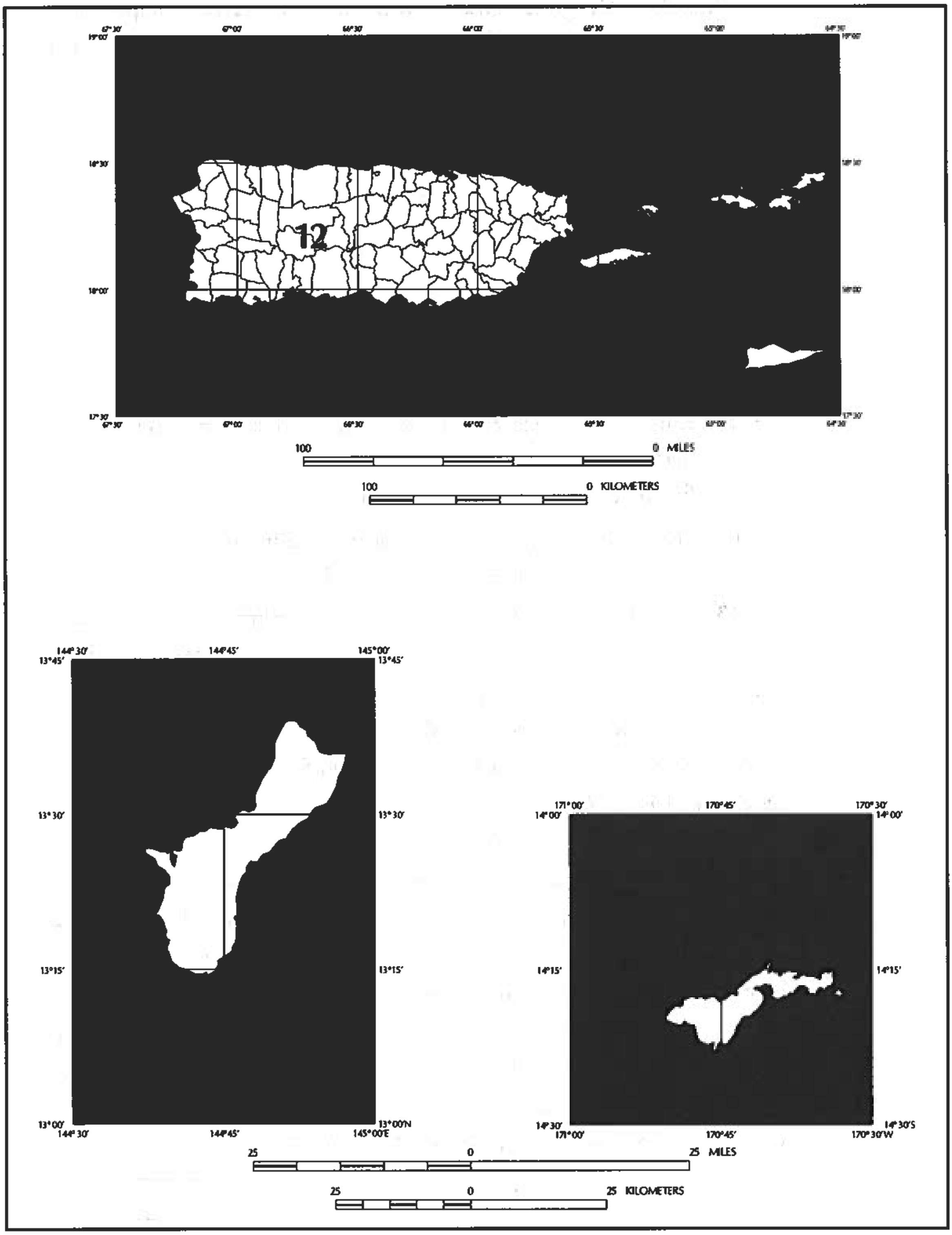

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Figure 19 Region-dependent transition period for longer-period ground motion T_L

13.2.7.3.2 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:

$$\text{For } T_c \leq T_L\text{: } S_{ac} = \frac{KS_{D1}}{T_c} \leq S_{DS} \qquad \text{(Eq 13-12)}$$

$$\text{For } T_c > T_L\text{: } S_{ac} = \frac{KT_L S_{D1}}{T_c^2} \qquad \text{(Eq 13-13)}$$

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.

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

13.2.8.1 General. When a site-specific procedure is specified or required by this standard to determine the MCE spectral response acceleration, 5 percent damped, at any period S_{aM}, the procedure shall comply with ASCE 7 Chapter 21, "Site-Specific Ground Motion Procedures for Seismic Design." The site-specific analysis shall be documented in a report. Refer to Sec. 13.2.4 for requirements for sites classified as Site Class F.

13.2.8.2 Design response spectrum.

13.2.8.2.1 Design response spectrum for impulsive components. The design response spectrum for impulsive components S_{ai} shall be based on 5 percent damping and Eq 13-14, except as noted. The design spectral response acceleration by Eq 13-14 shall not be less than 80 percent of the design spectral response acceleration by the general procedure (Sec. 13.2.7). 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. 13.2.7).

$$S_{ai} = US_{aM} \qquad \text{(Eq 13-14)}$$

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 by Eq 13-14 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 inches high and are not otherwise prevented from sliding laterally at least 1 inch.

See Sec. 13.5.2.1 and Sec. 13.5.2.2 for definitions of W_T and W_i.

13.2.8.2.2 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 13-15. The design spectral response acceleration by Eq 13-15 shall not be less than 80 percent of the design spectral response acceleration by the general procedure (Sec. 13.2.7).

$$S_{ac} = UKS_{aM} \qquad \text{(Eq 13-15)}$$

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. 13.2.8, except that the damping scaling factor K shall be set equal to 1.0.

13.2.9 *Horizontal design accelerations.*

13.2.9.1 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. 13.2.7) or, when specified or required, the site-specific procedure (Sec. 13.2.8). 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 seconds. The design acceleration shall be applied to the total weight of the tank and contents, unless the alternative procedure of Sec. 13.2.10 is used. The design acceleration shall be determined by the equation

$$A_i = \frac{S_{ai} I_E}{1.4R_i} \geq \frac{0.36 S_1 I_E}{R_i} \qquad \text{(Eq 13-16)}$$

Where:

A_i = design acceleration, stated as a multiple (decimal) of g

I_E = seismic importance factor from Table 24

R_i = response modification factor for the impulsive component from Table 28 for the type of structure

The other symbols have been previously defined in this section.

13.2.9.2 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. 13.2.7) or, when specified or required, the site-specific procedure (Sec. 13.2.8). The impulsive and convective design accelerations shall be determined by the following equations:

$$A_i = \frac{S_{ai} I_E}{1.4 R_i} \geq \frac{0.36 S_1 I_E}{R_i} \qquad \text{(Eq 13-17)}$$

$$A_c = \frac{S_{ac} I_E}{1.4 R_i} \qquad \text{(Eq 13-18)}$$

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 28 for the type of structure

The other symbols have been previously defined in this section.

13.2.10 *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 13-18.

4. Base shear including the effects of fluid–structure interaction or soil–structure interaction, or both, V_{alt} shall not be less than 75 percent of the base shear without the effects of fluid–structure interaction and soil–structure interaction V. When V_{alt} is less than $0.75V$, shears and overturning moments shall be scaled by the ratio $0.75V/V_{alt}$.

13.2.11 *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 75 percent of the base shear including the effects of fluid–structure interaction only V. When V_{alt} is less than $0.75V$, shears and overturning moments shall be scaled by the ratio $0.75V/V_{alt}$.

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:

$$X_{GC} = X_K X_C \qquad \text{(Eq 13-19)}$$

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

$$V = A_i W \qquad \text{(Eq 13-20)*}$$

Where:

V = lateral force, in pounds

A_i = design acceleration from Eq 13-16, 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 AISC, 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 20).

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.

* For equivalent metric equation, see Sec. 13.8.

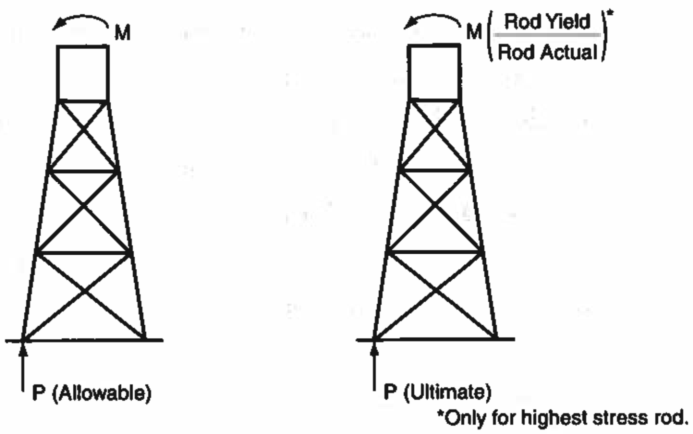


Figure 20 Diagram for checking foundation stability of cross-braced, column-supported elevated tanks

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 $0.14S_{DS}$. 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 square root sum of the squares (SRSS) method, unless otherwise specified.

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

$$V = A_i W \qquad \text{(Eq 13-21)*}$$

Where:*

V = lateral force, in pounds

A_i = design accelerations from Eq 13-16, stated as a multiple (decimal) of g

W = total weight of structure and contents, in pounds

* For equivalent metric equation, see Sec. 13.8.

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.

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 $0.14S_{DS}$. 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 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 $I_E/1.4R_i$ 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 $I_E/1.4R_i$ 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 ($2F_L \leq F_U$), 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

$$T_c = 2\pi\sqrt{\frac{D}{3.68g\tanh\left(\frac{3.68H}{D}\right)}} \qquad \text{(Eq 13-22)}$$

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

$$M_s = \sqrt{[A_i(W_sX_s + W_rH_t + W_iX_i)]^2 + [A_cW_cX_c]^2} \qquad \text{(Eq 13-23)*}$$

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 13-17, stated as a multiple (decimal) of g

A_c = convective design acceleration from Eq 13-18, stated as a multiple (decimal) of g

W_s = total weight of tank shell and significant appurtenances, in pounds

W_r = total weight of the tank roof, including framing and knuckle, plus permanent loads, if specified, in pounds

* For equivalent metric equation, see Sec. 13.8.

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 $D/H \geq 1.333$:

$$W_i = \frac{\tanh\left(0.866\dfrac{D}{H}\right)}{0.866\dfrac{D}{H}} W_T \qquad \text{(Eq 13-24)}$$

For $D/H < 1.333$:

$$W_i = \left[1.0 - 0.218\frac{D}{H}\right] W_T \qquad \text{(Eq 13-25)}$$

For all proportions of D/H:

$$W_c = 0.230\frac{D}{H}\tanh\left(\frac{3.67H}{D}\right) W_T \qquad \text{(Eq 13-26)}$$

Where:

W_T = total weight of tank contents, in pounds, determined by the equation

$$W_T = 62.4GH\left(\frac{\pi D^2}{4}\right) = 49GHD^2 \qquad \text{(Eq 13-27)*}$$

Where:

G = specific gravity = 1.0 for water

* For equivalent metric equation, see Sec. 13.8.

The 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 $D/H \geq 1.333$:

$$X_i = 0.375H \qquad \text{(Eq 13-28)}$$

For $D/H < 1.333$:

$$X_i = \left[0.5 - 0.094\frac{D}{H}\right]H \qquad \text{(Eq 13-29)}$$

For all proportions of D/H:

$$X_c = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \qquad \text{(Eq 13-30)}$$

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

$$V_f = \sqrt{[A_i(W_s + W_r + W_f + W_i)]^2 + [A_c W_c]^2} \qquad \text{(Eq 13-31)*}$$

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 13-23.

* For equivalent metric equation, see Sec. 13.8.

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

$$M_{mf} = \sqrt{[A_i(W_sX_s + W_rH_t + W_iX_{imf})]^2 + [A_cW_cX_{cmf}]^2} \qquad \text{(Eq 13-32)*}$$

For $D/H \geq 1.333$:

$$X_{imf} = 0.375\left[1.0 + 1.333\left[\frac{0.866\dfrac{D}{H}}{\tanh\left(0.866\dfrac{D}{H}\right)} - 1.0\right]\right]H \qquad \text{(Eq 13-33)}$$

For $D/H < 1.333$:

$$X_{imf} = \left[0.50 + 0.06\frac{D}{H}\right]H \qquad \text{(Eq 13-34)}$$

For all proportions of D/H:

$$X_{cmf} = \left[1.0 - \frac{\cosh\left(\dfrac{3.67H}{D}\right) - 1.937}{\dfrac{3.67H}{D}\sinh\left(\dfrac{3.67H}{D}\right)}\right]H \qquad \text{(Eq 13-35)}$$

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.

* For equivalent metric equation, see Sec. 13.8.

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

1. The overturning ratio J determined by Eq 13-36 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. 13.6 are satisfied.

The overturning ratio J is given by the equation

$$J = \frac{M_s}{D^2[w_t(1 - 0.4A_v) + w_L]} \qquad \text{(Eq 13-36)*}$$

Where:

J = overturning ratio

w_t = weight of the tank shell and portion of the roof reacting on the shell, determined by Eq 13-41, in pounds per foot of shell circumference

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

$$w_L = 7.9t_b\sqrt{F_yHG} \leq 1.28HDG \qquad \text{(Eq 13-37)*}$$

* For equivalent metric equation, see Sec. 13.8.

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

$$L = 0.216 t_b \sqrt{\frac{F_y}{HG}} \text{ in feet} \leq 0.035D \qquad \text{(Eq 13-38)*}$$

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

$$\sigma_c = \left[w_t(1 + 0.4A_v) + \frac{1.273M_s}{D^2}\right]\frac{1}{12t_s} \qquad \text{(Eq 13-39)*}$$

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

$$\sigma_c = \left[\frac{w_t(1 + 0.4A_v) + w_L}{0.607 - 0.18667J^{2.3}} - w_L\right]\frac{1}{12t_s} \qquad \text{(Eq 13-40)*}$$

* For equivalent metric equation, see Sec. 13.8.

In Eq 13-39 and Eq 13-40,

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

$$w_t = \frac{W_s}{\pi D} + w_{rs} \qquad \text{(Eq 13-41)*}$$

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 13-39.†

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

$$\sigma_s = \frac{\sqrt{N_i^2 + N_c^2 + (N_h A_v)^2}}{t_s} \qquad \text{(Eq 13-42)*}$$

For $D/H \geq 1.333$:

$$N_i = 4.5 A_i G D H \left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right] \tanh\left(0.866\frac{D}{H}\right) \qquad \text{(Eq 13-43)*}$$

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

$$N_i = 2.77 A_i G D^2 \left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right] \qquad \text{(Eq 13-44)*}$$

* For equivalent metric equation, see Sec. 13.8.

† The application of Eq 13-39 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 $D/H < 1.333$ and $Y \geq 0.75D$:

$$N_i = 1.39A_iGD^2 \tag{Eq 13-45}$$*

For all proportions of D/H:

$$N_c = \frac{0.98A_cGD^2 \cosh\left[\frac{3.68(H-Y)}{D}\right]}{\cosh\left[\frac{3.68(H)}{D}\right]} \tag{Eq 13-46}$$*

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 Section 3 or Section 14, as applicable. The allowable stress shall be reduced by the applicable joint efficiency (Table 15 or Section 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:

$$\sigma_e = 1.333\left(\sigma_a + \frac{\Delta\sigma_{cr}}{2}\right) \tag{Eq 13-47}$$

For mechanically anchored tanks:

$$\sigma_e = 1.333\sigma_a \tag{Eq 13-48}$$

Where:

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

* For equivalent metric equations, see Sec. 13.8.

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

$$\Delta\sigma_{cr} = \frac{\Delta C_c Et}{R} \qquad \text{(Eq 13-49)*}$$

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

$$\text{For } \frac{P}{E}\left(\frac{R}{t}\right)^2 \le 0.064{:}\Delta C_c = 0.72\left[\frac{P}{E}\left(\frac{R}{t}\right)^2\right]^{0.84} \qquad \text{(Eq 13-50)*}$$

$$\text{For } \frac{P}{E}\left(\frac{R}{t}\right)^2 > 0.064{:}\Delta C_c = 0.045 ln\left[\frac{P}{E}\left(\frac{R}{t}\right)^2 + 0.0018\right] + 0.194 \le 0.22 \qquad \text{(Eq 13-51)*}$$

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 $0.14S_{DS}$, 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. Sloshing shall be considered in determining the freeboard above the MOL. Freeboard is defined as the distance from the MOL to the lowest level of the roof framing. The freeboard provided shall meet the requirements of Table 29, unless otherwise specified. The sloshing wave height shall be determined by the equation

$$d = 0.5DA_f \qquad \text{(Eq 13-52)}$$

* For equivalent metric equations, see Sec. 13.8.

Table 29 Minimum freeboard requirements

	Seismic Use Group		
S_{DS}	I	II	III
$S_{DS} < 0.33g$	None	None	d
$S_{DS} \geq 0.33g$	None	$0.7d$	d

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 Seismic Use Groups I and II:

$$\text{When } T_c \leq 4: A_f = \frac{KS_{D1}I_E}{T_c} \tag{Eq 13-53}$$

$$\text{When } T_c > 4: A_f = \frac{4KS_{D1}I_E}{T_c^2} \tag{Eq 13-54}$$

For Seismic Use Group III:

$$\text{When } T_c \leq T_L: A_f = \frac{KS_{D1}}{T_c} \tag{Eq 13-55}$$

$$\text{When } T_c > T_L: A_f = \frac{KS_{D1}T_L}{T_c^2} \tag{Eq 13-56}$$

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-sec period, stated as a multiple (decimal) of g

I_E = seismic importance factor from Table 24

T_c = first mode sloshing wave period, in seconds

T_L = region-dependent transition period for longer-period ground motion, in seconds, shown in Figure 19

The other symbols have been previously defined in this section.

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 13-41). 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 AISC, increased one-third for seismic loading.

13.5.4.6 Sliding check. When a sliding check is specified, a coefficient of friction equal to tan 30° can be assumed. The allowable lateral shear shall be determined by the equation

$$V_{ALLOW} = \tan 30°[W_s + W_r + W_i + W_c](1 - 0.4A_v) \qquad \text{(Eq 13-57)*}$$

Where:

V_{ALLOW} = allowable lateral shear, in pounds

The other symbols have been previously defined in this section.

The allowable lateral shear shall be equal to or greater than the design shear at the top of the foundation due to horizontal design acceleration, V_f, determined by Eq 13-31 or additional shear resistance with a capacity of at least V_{NET} (see Sec. 3.8.7.1) must be provided.

Sec. 13.6 Piping Connections

13.6.1 *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 30 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 30 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 30 may be reduced to 70 percent of the values shown.

* For equivalent metric equation, see Sec. 13.8.

Table 30 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

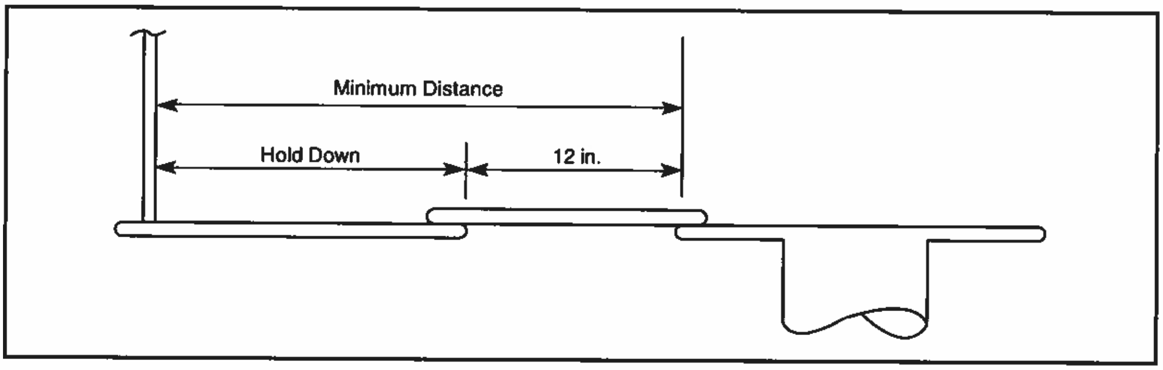


Figure 21 Bottom piping connection of a self-anchored, ground-supported flat-bottom tank

13.6.2 *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 13-38 plus 12 in. (305 mm), as shown in Figure 21.

Sec. 13.7 Foundation Design for Ground-Supported Flat-Bottom Tanks

13.7.1 *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 the ringwall and footing are designed to carry this eccentric loading. Water load shall not be used to reduce the anchor bolt load.

13.7.2 *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 13-39). This assumption does not permit larger response modification factors R_i and R_c than permitted for self-anchored tanks.

13.7.3 *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. 13.8 Equivalent Metric Equations

Metric equivalents of equations presented in Section 13 are as follows:

Equation Number	Equivalent Metric Equation	Variable	Metric Units
13-20	$V = 9.81A_iW$	V	N
13-21		W	kg
13-23	$M_s = 9.81\sqrt{[A_i(W_sX_s + W_rH_t + W_iX_i)]^2 + [A_cW_cX_c]^2}$	M_s	N-m
		W_s, W_r	kg
		W_i, W_c	kg
		X_s, H_t	m
		X_i, X_c	m
13-27	$W_T = 785.4GHD^2$	W_T	kg
		D, H	m
13-31	$V_f = 9.81\sqrt{[A_i(W_s + W_r + W_f + W_i)]^2 + [A_cW_c]^2}$	V_f	N
		W_s, W_r	kg
		W_f, W_i	kg
		W_c	kg
13-32	$M_{mf} = 9.81\sqrt{[A_i(W_sX_s + W_rH_t + W_iX_{imf})]^2 + [A_cW_cX_{cmf}]^2}$	M_{mf}	N-m
		W_s, W_r	kg
		W_i, W_c	kg
		X_s, H_t	m
		X_{imf}, X_{cmf}	m
13-36	$J = \dfrac{M_s}{D^2[w_t(1 - 0.4A_v) + w_L]}$	M_s	N-m
		D	m
		w_t, w_L	N/m

13-37	$w_L = 99 t_b \sqrt{F_y HG} \leq 201.1 HDG$	w_L	N/m
		t_b	mm
		F_y	MPa
		H,D	m
13-38	$L = 0.0172 t_b \sqrt{\frac{F_y}{HG}} \leq 0.035D$	L,D	m
		t_b	mm
		F_y	MPa
13-39	$\sigma_c = \left[w_t(1 + 0.4A_v) + \frac{1.273M_s}{D^2}\right]\left[\frac{1}{1{,}000t_s}\right]$	σ_c	MPa
		w_t	N/m
		M_s	N-m
		D	m
		t_s	mm
13-40	$\sigma_c = \left[\frac{w_t(1 + 0.4A_v) + w_L}{0.607 - 0.18667J^{2.3}} - w_L\right]\left[\frac{1}{1{,}000t_s}\right]$	σ_c	MPa
		w_t, w_L	N/m
		t_s	mm
13-41	$w_t = \frac{9.81\,W_s}{\pi D} + w_{rs}$	w_t, w_{rs}	N/m
		W_s	kg
		D	m
13-42	$\sigma_s = \frac{\sqrt{N_i^2 + N_c^2 + (N_h A_v)^2}}{1{,}000t_s}$	σ_s	MPa
		N_i, N_c, N_h	N/m
		t_s	mm
13-43	$N_i = 8{,}480 A_i GDH \left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right] \tanh\left(0.866\frac{D}{H}\right)$	N_i	N/m
		D,H,Y	m
13-44	$N_i = 5{,}220 A_i GD^2 \left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right]$	N_i	N/m
		D,Y	m
13-45	$N_i = 2{,}620 A_i GD^2$	N_i	N/m
		D	m
13-46	$N_c = \frac{1{,}850 A_c GD^2 \cosh\left[\frac{3.68(H - Y)}{D}\right]}{\cosh\left[\frac{3.68(H)}{D}\right]}$	N_c	N/m
		D,H,Y	m
13-49	$\Delta\sigma_{cr} = \frac{\Delta C_c E_t}{R}$	$\Delta\sigma_{cr}, E$	MPa
		t,R	mm
13-50	For $\frac{P}{E}\left(\frac{R}{t}\right)^2 \leq 0.064{:}\ \Delta C_c = 0.72\left[\frac{P}{E}\left(\frac{R}{t}\right)^2\right]^{0.84}$	P,E	MPa
		t,R	mm
13-51	For $\frac{P}{E}\left(\frac{R}{t}\right)^2 > 0.064{:}\ \Delta C_c = 0.045 ln\left[\frac{P}{E}\left(\frac{R}{t}\right)^2 + 0.0018\right] + 0.194 \leq 0.22$	P,E	MPa
		t,R	mm
13-57	$V_{ALLOW} = 9.81 \tan 30°[W_s + W_r + W_i + W_c](1 - 0.4A_v)$	V_{ALLOW}	N
		W_s, W_r	kg
		W_i, W_c	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 Section 3 and Section 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 Section 3 are superseded by any differing requirements of Section 14. Other requirements of Section 1 through Section 13 and Section 15 shall apply. Category 3 materials (see Table 33) shall not be used where the mapped spectral response acceleration at 1-sec period S_1 is greater than $0.15g$ and the mapped spectral response acceleration at 0.2-sec 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 ANSI/AWS B2.1 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 and ASTM 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 ASTM A517 steel, 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 ASTM A517 steel 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 material. These materials shall be limited to the thickness stated for the design metal temperatures shown in Tables 31 through 33, 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 category 2 materials may be used at thicknesses greater than tabulated for the design metal temperature provided the impact requirements specified in Sec. 14.1.4 and Sec. 14.1.5 are met, and provided 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 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 (including ornamental torus transition), the bottom plates, and other attachments and appurtenances not listed above shall conform to the requirements of Section 2.

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. ANSI/AWWA D100-11, including Section 14
2. Constructor
3. Year completed

4. Contract number
5. Nominal capacity, gal
6. Nominal diameter, ft
7. Top capacity level, ft
8. Design metal temperature
9. Shell material
10. Heat treatment

14.2.1 *Category 1.* Category 1 materials are listed in Table 31. 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 31. 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 32. 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 32. 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 33. The base-metal, weld-metal, and heat-affected zone shall be impact tested in accordance with Sec. 14.1.5. Category 3 materials shall not be used for tanks located in seismic zones 3 and 4.

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 22) 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, 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 the area of reinforcement is increased in inverse proportion to the

Table 31 Category 1 material requirements for shell plates in contact with water to be used for design metal temperature tabulated

Maximum Plate Thickness		Permissible Minimum Specifications		Maximum Insert Plate Thickness*	
in.†	*(mm)*	Specification No.	Grade	*in.*	*(mm)*
		+20°F (–6.7°C) or warmer			
½	(13)	ASTM A283	Gr C		
		ASTM A131	Gr A		
		ASTM A36			
1	(25)	ASTM A131	Gr B		
		ASTM A36, modified‡			
		CAN/CSA G40.21	Gr 44W, 38W		
		ASTM A442			
		–10°F (–23.3°C) or Warmer			
½	(13)	ASTM A131	Gr B		
		ASTM A36, modified‡			
		CAN/CSA G40.21	Gr 44W, 38W		
		ASTM A442			
1½	(38)	CAN/CSA G40.21	Gr 44T, 38T		
		ASTM A662	Gr B		
		ASTM A573	Gr 58		
		ASTM A516	Gr 60	A516 Gr 60, Norm 2½§	(64)
		–40°F (–40°C) or Warmer			
½	(13)	CAN/CSA G40.21	Gr 44T, 38T		
		ASTM A662	Gr B		
		ASTM A573	Gr 58		
		ASTM A516	Gr 60		
1½	(38)	ASTM A516	Gr 60 norm	2½	(64)
		ASTM A131	Gr CS norm	2	(51)
		CAN/CSA G40.21	Gr 44T, 38T norm		
		ASTM A662	Gr B norm	2	(51)
		ASTM A573	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, modified. Materials shall conform to ASTM A36 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 inches, 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.

Table 32 Category 2 materials

Group	ASTM Specification No.	Grade	Maximum Insert Thickness* in. (mm)	Lowest Design Metal Temperature for Shell Plates Without Impact Testing °F (°C)
1†	A573	Gr 70	—	+20 (–6.7) for $t \le 1\frac{1}{2}$ in. (38 mm)
	A588		—	+5 (–15.0) for $t \le 1$ in. (25 mm)
	A516	Gr 70	—	–10 (–23.3) for $t \le \frac{1}{2}$ in. (13 mm)
	A662	Gr C	—	
2‡	A573	Gr 70	—	–10 (–23.3) for $t \le 1\frac{1}{2}$ in. (38 mm)
	A516	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)
	A633	Gr C&D	2½ (64)	–20 (–28.9) for $t \le 1\frac{1}{2}$ in. (38 mm)
	A537	Cl 1	2½ (64)	–30 (–34.4) for $t \le 1$ in. (25 mm)
	A662	Gr C	—	–40 (–40) for $t \le \frac{1}{2}$ in. (13 mm)
3§	A537	Cl 2	2½ (64)	
	A678	Gr A	—	–40 (–40) for $t \le 1\frac{1}{2}$ in. (38 mm)
	A678	Gr B	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 33 Category 3 materials

ASTM Specification No.	Grade	Maximum Thickness in.	Maximum Thickness (mm)
A517	A, B,	1¼	(32)
	E, F, H	1½	(38)

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

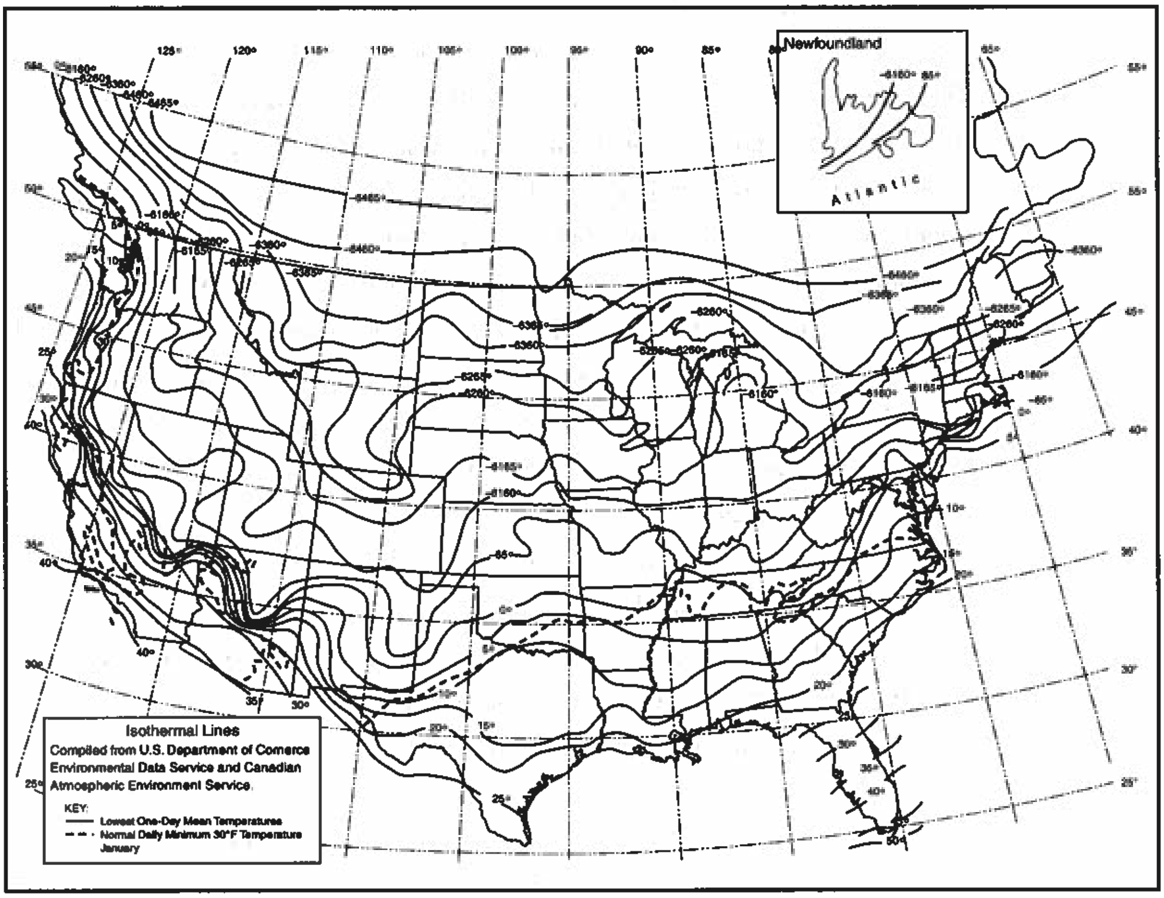


Figure 22 Isothermal lines for lowest one-day mean temperatures and normally daily minimum 30°F (–1.1°C) temperature line for January, United States and southern Canada

joints, provided (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 knuckle stiffening, 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:

* 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.

1. Prior to 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 hr after welding has been completed.

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 Section 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 Section 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 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, 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, 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, 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 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 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 or A181, 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 LF1 or LF2.

For all penetrations in shell courses constructed with category 3 material, only ASTM A592 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. Refer to Table 34 for 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.

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 31 and 32 for the material grade and design metal temperature, may be used, provided 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 31 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 the extra thickness is not used in any stress or wind stability calculation.

* Shell analysis theory may be ANSYS, Kalmin, STRUDL, or other commercially available finite element program.

Table 34 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	Specification for Structural Steel		1	19,330	133.3
ASTM A131	Specification for Structural Steel for Ships	A,B,CS	1	19,330	133.3
ASTM A516	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
CAN/ CSAG40.2	Structural Quality Steels	38W, 38WT	1	20,000	137.9
		44W, 44WT	1	21,670	149.4
ASTM A283	Specification for Low and Intermediate Tensile Strength Carbon Steel Plates, Shapes, and Bars	C	1	18,000	124.1
ASTM A662	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	Specification for Structural Carbon Steel Plates of Improved Toughness	58	1	19,200	132.4
		70	1	23,330	160.9
ASTM A588	Specification for High Strength Low Alloy Structural Steel With 50 ksi (345 MPa) Minimum Yield Point to 4 in. Thick	All	1	23,330	160.9
ASTM A633	Specification for Normalized High Strength Low Alloy Structural Steel	C,D	1	23,330	160.9
ASTM A678	Specification for Quenched and Tempered Carbon Steel Plates for Structural Applications	A	1	23,330	160.9
		B	1	26,670	183.9
ASTM A537	Specification for Pressure Vessel Plates, Heat Treated, Carbon Manganese Silicon Steel	Cl1	1	23,330	160.9
		Cl2	1	26,670	183.9
ASTM A517	Specification for Pressure Vessel Plates, Alloy Steel, High Strength, Quenched and Tempered	A,B, E,F,H	11B	38,330	264.3

14.3.2.5 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 Connection 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 around connections. The spacing of welds around connections shall conform to the following:

14.3.2.8.1 The outer weld toe of a nonstress-relieved weld around a penetration, around the periphery of a thickened insert plate, or around the periphery of a 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 prior to 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 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 35.

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

Table 35 Minimum thickness of bottom annulus

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	(179.3)	0.1875t*
ASTM A517 Material	38,330	(264.3)	0.50t*

*t is the bottom course thickness. In no case is the bottom annulus to be less than ¼ in. (6.4 mm) thick.

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 Appendix 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.

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 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 3/8 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 3/4 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.

4. Figure 23 illustrates the radiographic requirements specified in this section and is included for clarity.

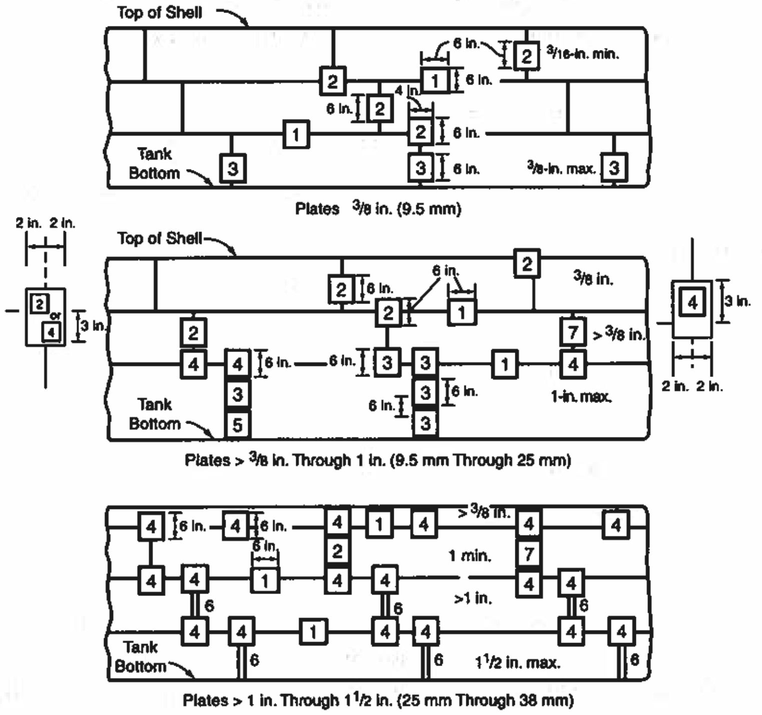
5. Plates shall be considered the same thickness when the thickness difference is 1/8 in. (3 mm) or less.

14.4.4 *Inspection of butt joints in annular plates.* When a butt-joint welded bottom annulus is required by Section 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. 11.9) shall be made on the second portion welded.



Numbers in the figure illustrate the following requirements: (1) horizontal radiograph, one in first 10 ft (3 m)—thereafter, each 100 ft (30 m) of joint; (2) vertical radiograph, one in first 10 ft (3 m)—thereafter, each 100 ft (30 m), 25 percent of which are to be at intersections; (3) vertical radiograph in each vertical seam in lowest course; (4) radiograph at all intersections of vertical with horizontal seams; (5) radiograph near the bottom of each vertical seam in lowest course; (6) seams fully radiographed; and (7) when shell rings above the bottom course exceed 3/4 in. (19 mm) in thickness, at least one radiograph in 25 percent of the vertical seams of shell rings more than 3/4 in. (19 mm) thick.

Figure 23 Radiographic requirements for tank shells according to Section 14

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 ANSI/AWS QC1.

Sec. 14.5 Certification of Compliance

A certification, in the form of Figure 24, shall be provided stating that the tank has been designed, fabricated, erected, inspected, and tested in accordance with Section 14 and that all such inspection and testing has been satisfactory.

CERTIFICATION OF COMPLIANCE WITH REQUIREMENTS OF 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 American Water Works Association Standard D100-11, entitled "AWWA Standard for Welded 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 all of 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 24 Certification of compliance with requirements of ANSI/AWWA D100, Section 14

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 the "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:

1. ACI† 355.1R-91—State-of-the-Art Report on Anchorage to Concrete.

* American National Standards Institute, 25 West 43rd Street, Fourth Floor, New York, NY 10036.
† American Concrete Institute, 38800 Country Club Drive, Farmington Hills, MI 48331.

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

3. ANSI/AWS Z49.1-05—Safety in Welding, Cutting and Allied Processes.

4. API† 653—Tank Inspection, Repair, Alteration, and Reconstruction, 4th Edition.

5. ASCE‡ 7-05—Minimum Design Loads for Buildings and Other Structures.

6. ASME§ BPVC-CC-BPV—BPVC Code Cases, 2010 Edition.

7. AWWA Manual M42—*Steel Water-Storage Tanks*, 1998 Edition.

8. IBC¶-09—International Building Code.

9. NFPA** 22-08—Standard for Water Tanks for Private Fire Protection.

10. NFPA 51B-09—Standard for Fire Prevention During Welding, Cutting, and Other Hot Work.

11. 1963. Technical Information Document 7024. *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).

12. 1971. *Earthquake Engineering for Nuclear Reactors*. San Francisco, Calif.: J.A. Blume & Associates.

13. Baker, E.H., et al. *Shell Analysis Manual*. NASA-CR-912. Washington, D.C.: National Aeronautic Association.

14. Baker, E.H., L. Kovalevsky, and F.L. Rish. 1972. *Structural Analysis of Shells*. New York, N.Y.: McGraw-Hill.

15. Housner, G.W. 1954. *Earthquake Pressures on Fluid Containers*. California Institute of Technology.

16. Malhotra, P.K., Wenk, T., and Wieland, M. *Simple Procedure for Seismic Analysis of Liquid-Storage Tanks*. Structural Engineering International, 3/2000.

17. Miller, C.D. 1996. Effects of Internal Pressure on Axial Compressive Strength of Cylinders and Cones.

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

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

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

§ ASME International, Three Park Avenue, New York, NY 10016.

¶ International Code Council, 500 New Jersey Avenue NW, 6th Floor, Washington, DC 20001.

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

18. 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.

19. Veletsos, A.S., and J.Y. Yang. 1976. *Dynamics of Fixed-Base Liquid Storage Tanks.* Houston, Texas: Rice University.

20. 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 rods, the nut requirements of ASTM A194 typically apply.

SECTION A.3: GENERAL DESIGN (Refer to Sec. 3 of ANSI/AWWA D100)

Sec. A.3.1 Design Loads

A.3.1.4 *Wind load.* In preparing this standard, the Revision Task Force and Committee primarily used ASCE 7-05 as the basis for the minimum wind design loads. In Eq 3-1, 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.

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}.

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 ASCE 7 has increased the load factor on wind from 1.3 to 1.6 for ultimate strength design. This was done in part because a wind directionality factor of 0.85 was built into the 1.3 load factor. By assuming that K_d = 1.0, the calculated wind loads may be used with load factors that have not been adjusted upward to account for the removal of this factor.

The gust-effect factor G accounts for along-wind loading effects caused by wind turbulence–structure interaction. This standard uses a gust-effect factor of 1.0 and is considered appropriate for all elevated tank styles. This value may be conservative for tanks that can be classified as rigid structures, such as single-pedestal tanks with large pedestal diameters. The gust-effect factor may be evaluated per ASCE 7 guidelines and reduced to a more appropriate value less than 1.0, but not less than 0.85. The gust-effect factor is not intended to account for across-wind loading effects, vortex shedding, instability caused by galloping, or flutter of dynamic torsional effects. Tall, slender tanks should also be evaluated for these effects using the recognized literature referenced in ASCE 7.

Sec. A.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 6 of Sec. A.1.4 may be used to design shells subjected to biaxial compression or biaxial compression–tension (circumferential membrane stress is compressive).

The alternative procedures—Method 2 and Method 3—for determining the allowable local buckling compressive stress F_L are based on analysis and include the stabilizing effect of hydrostatic pressure. Method 2 and Method 3 only apply to water-filled shells having thickness-to-radius ratios greater than 0.001 and less than or equal to 0.003 and $(t/R)_c$ for Methods 2 and 3, respectively. The 0.0010 limit represents a practical lower limit. The stabilizing effect of hydrostatic pressure has little effect when the thickness-to-radius ratio is greater than $(t/R)_c$.

Experimental investigations (see reference 17 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 diamondshaped 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 t/R 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 wave length 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 3-20 and Eq 3-21 apply. When e equals $0.08\sqrt{Rt}$, the values of C_o are 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. Eq 3-22 and Eq 3-23 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. The procedure for determining the allowable local buckling compressive stress given in ANSI/AWWS D100-96 has been renamed Method 1. Method 1 is mandatory for shells that do not contain water.

A.3.4.3.2 Method 2. There are no additional analysis requirements for Method 2. Method 2 allowables are determined, in general, by applying a 25 percent increase to allowables obtained by Method 1. A transition curve is used at the beginning of the region from elastic to inelastic buckling. The hydrostatic pressure has an influence on the percentage increase in this region. It has been shown that the Method 2 allowables are less than would be obtained by relaxing the shell tolerance, e_x, to $0.06\sqrt{Rt}$, a value 1.5 times the Method 3 tolerance level. For this comparison study, the procedure to determine the allowable compressive stress, F_L, would use Eq 3-19 through Eq 3-33 for Method 3, except that Eq 3-20 and Eq 3-21 become Eq A.3-1 and Eq A.3-2, respectively:

$$C_o = \frac{76.65}{195 + \frac{R}{t}} \quad \text{for } t/R > 0.00161 \qquad \text{(Eq A.3-1)}$$

$$C_o = 0.09375 \text{ for } t/R \leq 0.00161 \qquad \text{(Eq A.3-2)}$$

Excessive translation and/or rotation at the lower boundary of the actual structure may reduce the shell buckling capacity in the boundary region to values less than theoretical. Accordingly, Method 2 excludes shell sections within a distance of $4\sqrt{Rt}$ from the lower boundary.

A.3.4.3.3 Method 3. Method 3 equations are based on shells with maximum deviation e equal to $4\sqrt{Rt}$. 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 3-18 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 17 of Sec. A.1.4 (Eq 3-19), 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 due to 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.3.6.1.7 Maximum rafter spacing. Eq 3-39 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.

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.3.8.5 *Anchor requirements.* Anchors other than anchor bolts and anchor straps may be used provided 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.3.8.5.1 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.3.8.7 *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.

A.3.8.9 *Design loads.* Loads associated with the minimum operating level of water may be used to resist uplift forces for ground-supported flat-bottom tanks. The minimum operating level is usually the invert of the inlet/outlet pipe for ground-supported flat-bottom tanks.

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 salt water 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. 4 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 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.

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.

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

Sec. A.5.4 Access

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 Section 8, Welding; Section 10, Erection; and Section 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, Section 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, Section 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

The use of dissimilar metals is not recommended within the tank water container to reduce the potential for long-term, and possibly significant, galvanic corrosion. When dissimilar metals are installed inside the tank below the top capacity level (TCL), they should be electrically isolated from the carbon steel tank components to which they are attached.

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 prior to 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

A7.4.2.3 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

The use of dissimilar metals is not recommended within the tank water container to reduce the potential for long-term, and possibly significant, galvanic corrosion. When dissimilar metals are installed inside the tank below the TCL, they should be electrically isolated from the carbon steel tank components to which they are attached.

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.* Welds, including fillet welds and partial joint penetration welds, will be qualified in accordance with the applicable rules of ASME BPVC Sec. IX or ANSI/AWS B2.1. AWS standard welding procedure specifications may be used under either the ASME BPVC Sec. IX rules or the ANSI/AWS D1.1 rules.

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 $1/16$ in. (1.6 mm) in order to maintain the strength of the weld. The maximum root opening of $3/16$ in. (4.8 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 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 19 may be used as a guide for determining if rolling is recommended.

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 why 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.

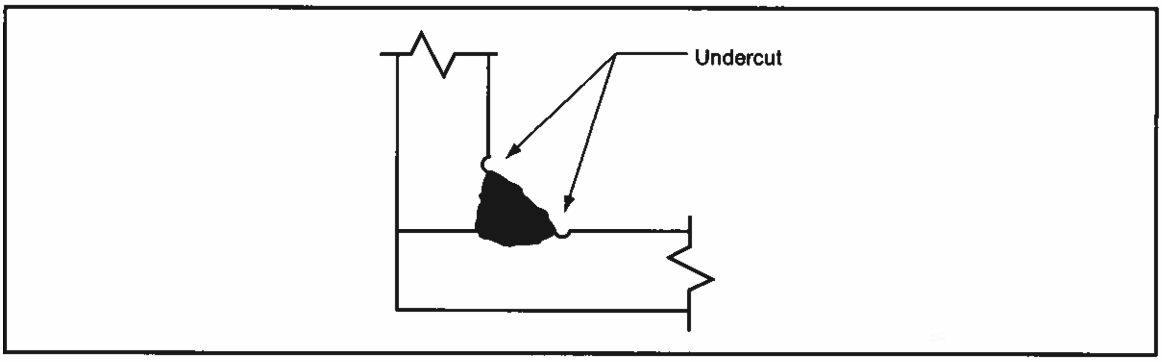


Figure A.1 Typical Undercut

A.11.4.2 *Visual inspection.* Additional information related to visual inspection can be found in ANSI/AWS D1.1, Sec. 5.24 and Figure 5.4, and in ANSI/AWS 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.1.

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 corrosion protection.

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 aboveground.* 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 7-05.

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 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:

$$S_S = 2.5S_P \qquad \text{(Eq A.11-1)}$$

$$S_1 = 1.25S_P \qquad \text{(Eq A.11-2)}$$

Where:

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

S_1 = mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1-sec 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. 13.2.7 or the site-specific procedure of Sec. 13.2.8. The general procedure in which spectral response acceleration parameters for the maximum considered earthquake ground motions are derived using Figures 5 through 18, modified by site coefficients to include local site effects, and scaled to design values, are permitted to be used for any structure except as specifically indicated in the standard. The site-specific procedure of Sec. 13.2.8 is permitted to be used for any structure and shall be used when specified or required by the standard.

A.13.2.1 *Seismic Use Group.* Tanks are classified in the appropriate Seismic Use Group based on the function and hazard to the public. A higher Seismic Use Group may be specified to match the risk management approach for the tank or facility. Specifying a higher Seismic Use Group increases the seismic importance factor I_E, used to define the design acceleration, and indirectly influences the performance level expected of the tank. Selection of the appropriate Seismic Use Group 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 Seismic Use Group for use with this standard.

A.13.2.1.1 Seismic Use Group III. Tanks serving the following types of applications may be assigned Seismic Use Group III, unless an alternative or redundant source is available:

Table A.1 Seismic use group

	Seismic Use Group		
Occupancy Category	I	II	III
I	X		
II	X		
III		X	
IV			X

1. Fire, rescue, and police stations.

2. Hospitals and emergency treatment facilities.

3. Power-generating stations or other utilities required as emergency backup facilities for Seismic Use Group III 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).

A.13.2.1.2 Seismic Use Group II. Tanks serving the following types of applications may be assigned Seismic Use Group II, unless an alternative or redundant source is available:

1. Power-generating stations and other public utility facilities not included in Seismic Use Group III, but required for continued operation.

2. Water and wastewater treatment facilities required for primary treatment and disinfection for potable water.

A.13.2.1.3 Seismic Use Group I. Seismic Use Group I is used for tanks not designated Seismic Use Group III or Seismic Use Group II.

A.13.2.4 *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.

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.

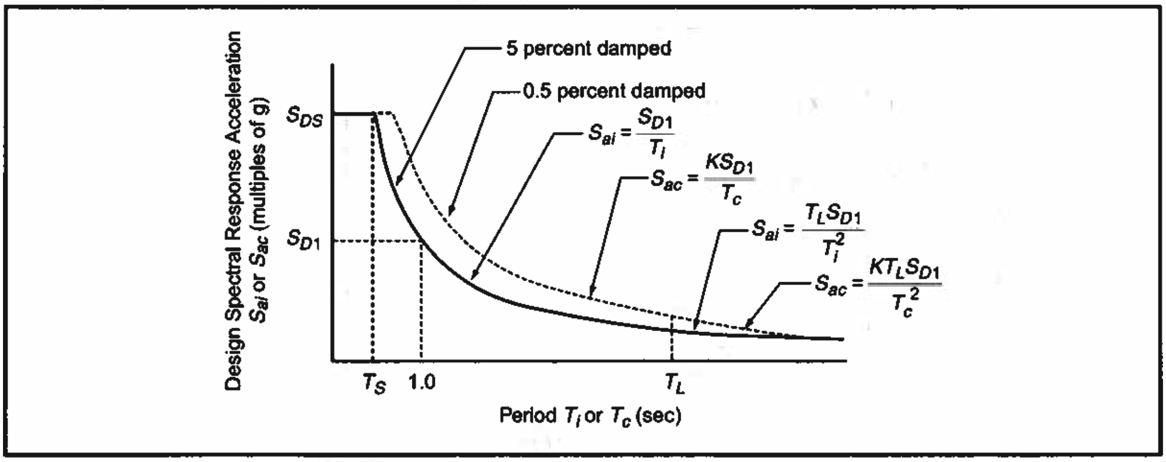


Figure A.2 Design response spectra—general procedure

A.13.2.7.3 Design response spectra. The design response spectra for the general procedure (Eq 13-9 through Eq 13-13) are shown in Figure A.2.

A.13.2.8 *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. 13.2.7, and is required for tanks located on Site Class F soils.

A.13.2.8.2 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-sec period, except that it shall not be taken less than 90 percent of the peak spectral acceleration at any period larger than 0.2 seconds. Similarly, the parameter S_{D1} shall be taken as the greater of the spectral acceleration at 1-sec period or two times the spectral acceleration at 2.0-sec 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. 13.2.7. The resulting site-specific design spectrum should be generated in accordance with Sec. 13.2.7.3.1 and should be smoothed to eliminate extreme humps and jagged variations.

A.13.2.8.2.1 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

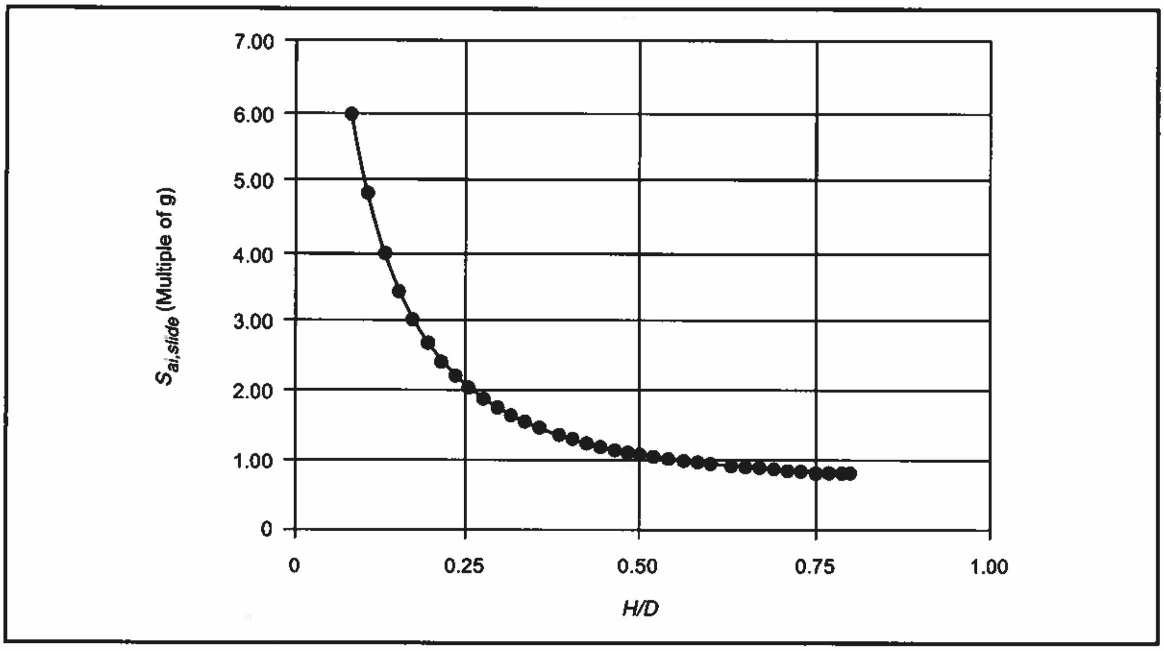


Figure A.3 Design spectral response acceleration that causes the tank to slide $S_{ai,slide}$

$$S_{ai,slide} = \tan 30° \left(\frac{W_T}{W_i} \right) \qquad \text{(Eq A.11-3)}$$

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 13-27

W_i = weight of effective mass of tank contents that moves in unison with the tank shell, in pounds, determined by Eq 13-24 or Eq 13-25

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 for bottom plates placed on concrete or cushions constructed of sand, crushed rock, or asphaltic road mix. A plot of the above equation is shown in Figure A.3.

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.

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.

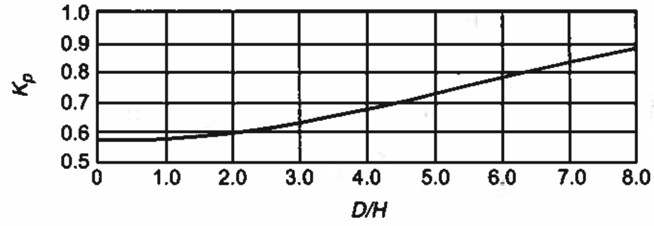


Figure A.4 Curve for obtaining factor K_p for the ratio D/H

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.

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 16, 18, or 19 of Sec. A.1.4.

The first mode sloshing wave period T_c may be determined by Eq 13-22, or by the graphical procedure using Eq A.11-4 and Figure A.4.

$$T_c = K_p\sqrt{D} \qquad \text{(Eq A.11-4)}$$

Where:

T_c = first mode sloshing wave period, in seconds

K_p = factor from Figure A.4 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 13-23) 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 Eq 13-24 through Eq 13-26, or by multiplying W_T by the ratios W_i/W_T and W_c/W_T obtained from Figure A.5. 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 Eq 13-28 through Eq 13-30, or by multiplying H by the ratios X_i/H and X_c/H obtained from Figure A.6.

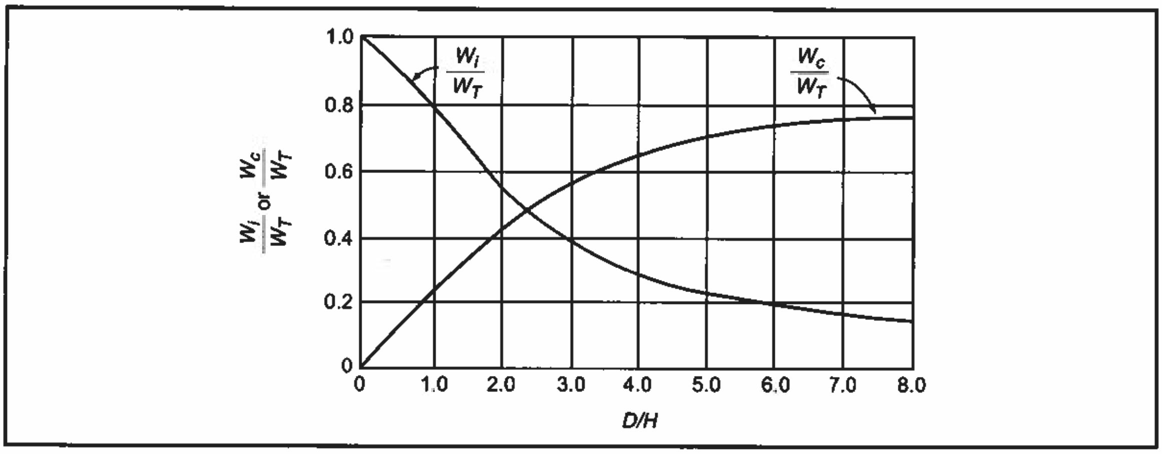


Figure A.5 Curves for obtaining factors W_i/W_T and W_c/W_T for the ratio D/H

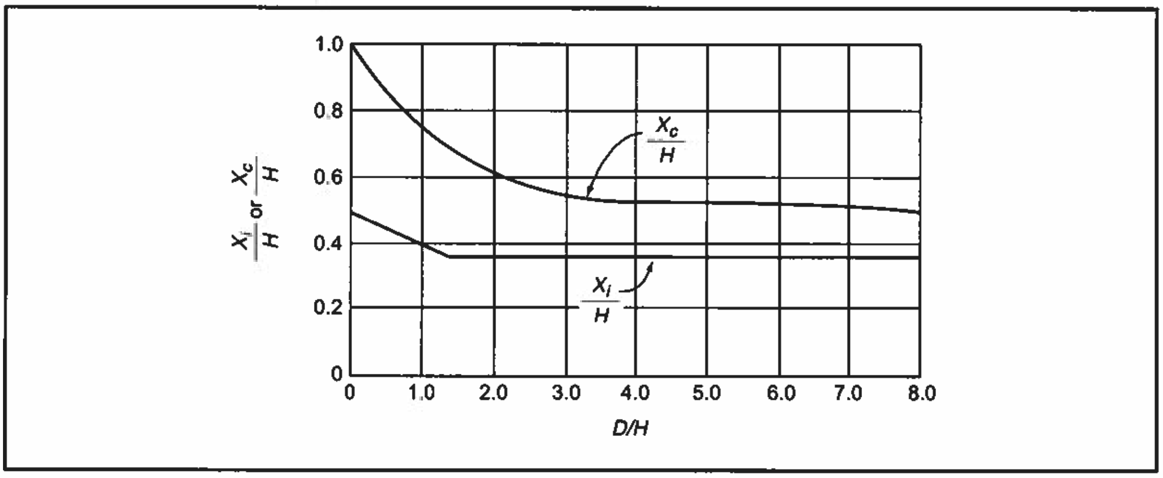


Figure A.6 Curves for obtaining factors X_i/H and X_c/H for the ratio D/H

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 13-23). 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 13-32) 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 Eq 13-33 through Eq 13-35.

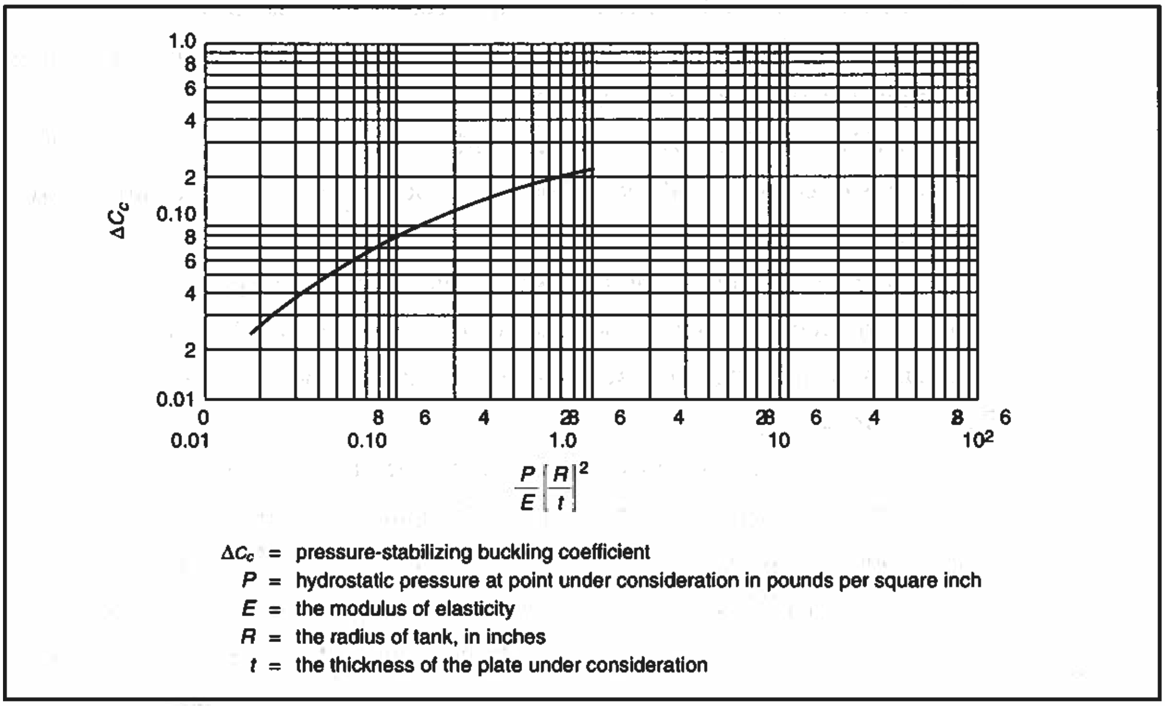


Figure A.7 Pressure-stabilizing buckling coefficient ΔC_c for self-anchored tanks

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 Eq 13-50 and Eq 13-51, or obtained from Figure A.7.

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.

A.13.5.4.4 Freeboard. If freeboard for the sloshing wave is not specified, some loss of contents and roof damage may occur if the tank is completely full during an earthquake.

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

$$y_u = \frac{F_y L^2}{83{,}300 t_b} \qquad \text{(Eq A.11-5)}$$

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 13-38

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

Section 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 (87.0 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 Section 3 designs has been sufficiently low that brittle fractures have not occurred and the Section 3 design philosophy has resulted in safe structures.

Steels with controlled chemical and alloy compositions that justify higher tensile working stresses are available. Section 14 provides an alternative design basis to that specified in Section 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 Section 14 are comparable to working stress levels allowed in other steel construction industries (allowable stress design provisions of AISC).

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 Section 14 tanks will have compared with the same size tank built to Section 3 design stresses. Because the Section 3 tank will be thicker than its Section 14 counterpart, the Section 3 tank has inherently higher resistance to these loads. Selection of proper seismic design level is equally important to both Section 3 and Section 14 tanks. Generally, Section 14 designs are suitable for tanks over 1,000,000 gallons (3,785.4 m^3) nominal capacity.

A.14.1.1 *Applicability.* When Section 14 is used, the requirements of Sec. 2.2.3.2 are superseded in addition to those of Section 3. Other requirements of the standard still apply, except where superseded by specific provisions of Section 14.

A.14.1.4 *Welding procedure qualification.* When Section 14 design is used, notch tough 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 material, may be found in API 650.

A.14.2.6.3 (6) Temporary attachments to shell courses of any Section 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.

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
1.2.7.1	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.
1.2.7.2	Maximum operating level (MOL)	The MOL shall be taken as the TCL.
1.3	Details of welded joints	Furnishing details of welded joints is not required.
2.1	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.
3.1.4.1	Basic wind speed for special wind regions	The basic wind speed for the general area, shown in Figure 1, will be used for special wind regions.
3.1.4.2	Velocity pressure exposure coefficient	Wind loads shall be based on velocity pressure exposure coefficients for Exposure C.
3.1.4.5	Projected area for wind loads on shrouds	The projected area of the shroud shall be the same height as 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.
3.5.2	Shell thickness for intermediate shell girders	The average, as-ordered shell thicknesses shall be used to determine height *h* in Eq 3-36.
3.6.1.6	Painting of contract 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.
4.4.3.1	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 3/8 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 8 ft (2.4 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.
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 3/8 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 8 ft (2.4 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.
12.3.1.1	Safety factor for gravity loads plus wind load for cross-braced multicolumn tanks	The minimum safety factor for gravity loads plus wind load shall be 3.0. Exception: The safety factor may be reduced to 2.25 when specified in the geotechnical report.
12.3.2.1	Safety factor for gravity loads plus wind load for single-pedestal tanks, standpipes, and reservoirs	The minimum safety factor for gravity loads plus wind load shall be 3.0. Exception: The safety factor may be reduced to 2.25 when specified in the geotechnical report.
12.4.7	Tolerances on anchor installation for cross-braced multicolumn tanks	The design of anchors and anchor attachments shall accommodate and installation shall comply with the tolerances given in Sec. 12.4.7.

12.5.3	Tolerances on anchor installation for single-pedestal tanks	The design of anchors and anchor 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 aboveground	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. 13.2.7) or, if required by the standard, the site-specific procedure (Sec. 13.2.8).
13.2.1	Seismic Use Group	Seismic Use Group III shall be used.
13.2.4	Site Class	Site Class D shall be used when the soil properties are not known in sufficient detail to determine the Site Class.
13.2.8.1	Site-specific procedure	The site-specific procedure must be used only if required by the standard.
13.2.9.1	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. 13.2.7) or, if required by the standard, the site-specific procedure (Sec. 13.2.8).
13.2.9.2	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. 13.2.7) or, if required by the standard, the site-specific procedure (Sec. 13.2.8).

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.
13.5.4.4	Freeboard requirements for ground-supported flat-bottom tanks	The freeboard provided shall meet the requirements of Table 29.
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.
13.6.1	Piping flexibility	The piping system shall accommodate the design displacements given in Table 30.
14.1.1	Applicability of Section 14	The design basis shall be Section 3.
14.2.4	Design metal temperature	The design metal temperature shall be the lowest one-day mean ambient temperature from Figure 22 plus 15°F (–9°C).
14.4.5	Certified welding inspector	A certified welding inspector is not required.
15.3	Thermal expansion	The connection of the roof to the tank shall be capable of withstanding thermal expansion caused by a temperature range of –40°F to +140°F (–40°C to +60°C).
15.4.1	Aluminum finish	Aluminum dome roof materials shall have a mill finish.
15.4.5.1	Sealants	Sealants shall be silicone compounds conforming to ASTM C920.
15.4.5.2	Gaskets	Preformed gasket material shall be made of silicone or neoprene.

15.6.3.3	Wind loads	Wind load shall be based on a wind velocity of 100 mph (45 m/sec).
15.7.2	Separation of carbon steel and aluminum	Aluminum shall be isolated from the carbon steel by an austenitic stainless-steel spacer or an elastomeric isolator bearing pad.
15.8	Minimum dome spherical radius	The minimum dome spherical radius shall be 0.7 times the tank diameter.
15.10.2	Welding	Structural welding of aluminum may be performed after field erecting of the dome.
15.10.2	Examination and qualification records	A full set of satisfactory examination and qualification records prior to field erection is not required.

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