Specification for Safety-Related Steel Structures for Nuclear Facilities

January 31, 2012

Supersedes the Specification for Safety-Related Steel Structures for Nuclear Facilities dated September 20, 2006 and all previous versions of this specification

Approved by the AISC Committee on Specifications
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by

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PREFACE

The AISC Specification for Safety-Related Steel Structures for Nuclear Facilities, hereafter referred to as the Nuclear Specification, addresses the design, fabrication and erection of safety-related steel structures for nuclear facilities. This document uses the 2010 AISC Specification for Structural Steel Buildings, hereafter referred to as the Specification, as the baseline document and modifies the specific portions of the Specification to make it applicable to the design, fabrication and erection of safety-related steel structures for nuclear facilities.

Safety-related steel structures in nuclear facilities, which provide support and protective functions to equipment vital to the facility, are subjected to certain unique design forces and loads resulting from postulated accidents (such as turbine generated missiles and jet forces from high energy line breaks) and from extreme natural phenomena (tornadoes and earthquakes). The relevant regulatory and jurisdictional authorities (for example, the Nuclear Regulatory Commission and the Department of Energy) dictate special quality assurance requirements and additional design requirements associated with these structures. As such, safety related nuclear structures require special design provisions. The provisions specified herein are to be used in conjunction with the Specification. The Nuclear Specification consists of modifications (additions, deletions and replacements) to the Specification.

The Nuclear Specification has been developed as a consensus document to provide uniform practice in the design of steel-framed structures for nuclear facilities. This specification was approved by the AISC Committee on Specifications:

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A nonmandatory Commentary provides background for the Nuclear Specification provisions and the user is encouraged to consult it.
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<th>Definition</th>
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<td>$C$</td>
<td>Rated capacity of crane.</td>
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<td>$D$</td>
<td>Dead loads due to the weight of the structural elements, fixed-position equipment, and other permanent appurtenant items; weight of crane trolley and bridge</td>
<td>NB2.1</td>
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<tr>
<td>$E_o$</td>
<td>Loads generated by the operating basis earthquake</td>
<td>NB2.2</td>
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<tr>
<td>$E_s$</td>
<td>Loads generated by the safe shutdown or design basis earthquake</td>
<td>NB2.3</td>
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<td>$L$</td>
<td>Live load due to occupancy and moveable equipment, including impact</td>
<td>NB2.1</td>
</tr>
<tr>
<td>$L_r$</td>
<td>Roof live load</td>
<td>NB2.1</td>
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<td>$P_a$</td>
<td>Maximum differential pressure load generated by the postulated accident</td>
<td>NB2.4</td>
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<td>Required compressive strength using ASD load combinations, kips (N)</td>
<td>Table NB3.2</td>
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<td>$R$</td>
<td>Rain load</td>
<td>NB2.1</td>
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<tr>
<td>$R_a$</td>
<td>Pipe and equipment reactions generated by the postulated accident, including $R_p$.</td>
<td>NB2.4</td>
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<tr>
<td>$R_p$</td>
<td>Pipe reactions during normal operating, start-up, or shutdown conditions, based on the most critical transient or steady-state condition</td>
<td>NB2.1</td>
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<tr>
<td>$S$</td>
<td>Snow load as stipulated in ASCE/SEI 7 for Category IV facilities</td>
<td>NB2.1</td>
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<td>Thermal loads generated by the postulated accident, including $T_o$.</td>
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<td>Wind load as stipulated in ASCE/SEI 7 for Category IV facilities</td>
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<td>$W_t$</td>
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<td>Jet impingement load generated by the postulated accident</td>
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<tr>
<td>$Y_r$</td>
<td>Loads on the structure generated by the reaction of the broken high-energy pipe during the postulated accident</td>
<td>NB2.4</td>
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<tr>
<td>$Y_m$</td>
<td>Missile impact load, such as pipe whipping generated by or during the postulated accident</td>
<td>NB2.4</td>
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<td>$\varepsilon_{sf}$</td>
<td>Strain corresponding to the onset of strain hardening</td>
<td>Table NB3.1</td>
</tr>
<tr>
<td>$\varepsilon_{uf}$</td>
<td>Strain corresponding to elongation at failure (rupture)</td>
<td>Table NB3.1</td>
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<tr>
<td>$\varepsilon_y$</td>
<td>Strain corresponding to yield stress</td>
<td>Table NB3.1</td>
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<tr>
<td>$\mu$</td>
<td>Ductility factor</td>
<td>NB3.14</td>
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GLOSSARY

The terms listed below shall be used in addition to or replacements for those in the AISC Specification for Structural Steel Buildings. Glossary terms are italicized where they first appear in a sub-section (i.e., A1, A2) of the Nuclear Specification.

Authority having jurisdiction (AHJ). Federal government agency (or agencies), such as the Nuclear Regulatory Commission or the Department of Energy, that is empowered to issue and enforce regulations affecting the design, construction and operation of nuclear facilities.

Certificate of compliance. Document written by the fabricator to affirm that the material was procured, dedicated, fabricated, coated, inspected and documented in accordance with the requirements of the standard and the contract documents.

Certified material test report (CMTR). Document identifying the chemical analysis, physical test data, and any other testing necessary to show compliance of the item for which the CMTR is supplied.

Dedication. The process by which material that is obtained from a commercial source is validated to be used in safety-related applications. In this process the critical characteristics for design are identified in the dedication plan and tested by an approved lab.

Design basis earthquake (or) Design/Evaluation Basis Earthquake (DBE). See safe shutdown earthquake (SSE). Term used in connection with DOE facilities; also used interchangeably for older nuclear power facilities.

Ductility factor. Ratio of permitted strain to the strain at yield or deformation to the deformation at yield.

Engineer of record. Individual or organization, designated by the owner, responsible for the preparation of the plans and specifications for the nuclear facility structures, or for the evaluation of the existing structure(s). The engineer of record as an individual or part of an organization shall be a licensed professional engineer, qualified to fulfill the assigned responsibility.

Impactive force. Time-dependent loads due to collision of masses that are associated with finite amounts of kinetic energy. The impactive load is determined by the inertia and stiffness properties of the impactor and the target structure. Impactive loads include the following examples/types: tornado-borne missiles, whipping pipes, aircraft missiles, and other internal and external missiles.

Impulsive force. Time-dependent loads which are not associated with collision of solid masses. The loads are not dependent on the target mass or stiffness properties. Impulsive loads include the following examples/types: jet impingement load, blast pressure, compartment pressurization, and jet shield reactions.

Jet impingement load. Force-time history depicting the forces resulting from the direct strike by a dense, high velocity jet of steam or water onto a structure, system or component.
Jet shield. Device used to protect adjacent structures, systems or components from the effects of a dense, high velocity jet of steam or water, resulting from the rupture of a high energy pipe line.

Missile impact. Collision of a projectile [for example, tornado-borne missile (see definition) or plant-generated missile] with a structure, system or component.

Operating basis earthquake (OBE). Earthquake that could reasonably be expected to occur at the plant site during the operating life of the plant considering the regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake that produces the vibratory ground motion for which the features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional. An earthquake greater than the OBE is associated with plant shutdown and inspection. See Appendix S of 10CFR50.

Owner. Organization responsible for the design, construction, operation, maintenance and safety of the nuclear facility.

Pipe whip impact barrier. Energy absorbing element used to protect safety-related structures, systems or components from the potentially damaging forces of a whipping, high energy pipe.

Pipe whip restraint. Energy absorbing device used to limit the potentially damaging motion of a whipping, high energy pipe, resulting from a pipe break/rupture, through the confining effects of the device.

Quality assurance (QA). In safety-related work, the program identifying the planned or systematic actions necessary to provide adequate confidence that an item or facility will be designed, fabricated, erected or constructed in accordance with the plans and specification.

Quality assurance inspector (QAI). Individual designated to provide quality assurance inspection for the work being performed.

Quality control (QC). In safety-related work, a process employed by the fabricator, erector or constructor to verify that the item or facility is fabricated, erected or constructed in accordance with the plans and specification.

Safe shutdown earthquake (SSE). Earthquake that produces the vibratory ground motion for which certain structures, systems and components in the nuclear power plant must be designed to remain functional (see Appendix S of 10CFR50). In the DOE nuclear facilities and older nuclear power plants, Design Basis Earthquake (or Design/Evaluation Basis Earthquake) (DBE) is used conveying the same meaning as SSE for design purposes.

Safety-related. Classification that applies to structures, systems or components used in a nuclear power plant that are relied upon during or following design basis events to assure:

1. The integrity of the reactor coolant pressure boundary;
2. The capability to shut down the reactor and maintain it in a safe shut down condition; or
3. The capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10CFR100.
Specified design (basis) tornado. Combination of translational speed, rotational speed, and prescribed pressure drop related to the environmental effects of a tornado (as defined by the licensing basis, design basis, and/or regulatory requirements; for example, USNRC Regulatory Guide 1.76).

Tornado-borne missiles. Missiles of specific weight and velocity (as defined by the AHJ for the facility site) and assumed to impact structures after becoming airborne as a result of tornado winds and pressures.
CHAPTER NA
GENERAL PROVISIONS

Modify Chapter A of the Specification as follows.

Replace preamble with the following:

This chapter states the scope of the Specification for Safety-Related Steel Structures in Nuclear Facilities, summarizes referenced specification, code and standard documents, and provides requirements for materials and design documents.

The chapter is organized as follows:

NA1. Scope
NA2. Referenced Specifications, Codes and Standards
NA3. Material
NA4. Structural Design Drawings and Specifications
NA5. Quality Assurance

NA1. SCOPE

Replace section with the following:

The Specification for Safety-Related Steel Structures in Nuclear Facilities, hereafter referred to as the Nuclear Specification, shall apply to the design of safety-related steel structures, and steel elements in nuclear facilities.

The Chapter, Appendix and Section designations within the Nuclear Specification are preceded by letter N to denote nuclear facility provisions.

The Nuclear Specification is compatible with the AISC Specification for Structural Steel Buildings (ANSI/AISC 360), hereafter referred to as the Specification. Provisions of the Specification are applicable unless stated otherwise. Only those sections that differ from the Specification provisions are indicated in the Nuclear Specification.

The Nuclear Specification includes the list of additional Symbols, additional Glossary terms, Chapters NA through NN and Appendices N1 through N8. The Commentary and User Notes interspersed throughout the Nuclear Specification are not part of the Nuclear Specification.

User Note: User notes are intended to provide concise and practical guidance in the application of the provisions.

The steel elements shall be as defined in the AISC Code of Standard Practice for Steel Buildings and Bridges, Section 2.1, hereafter referred to as the Code of Standard Practice.
Structures and structural elements subject to the Nuclear Specification are those steel structures which are part of a safety-related system or which support, house or protect safety-related systems or components, the failure of which would impair the safety-related functions of these systems or components. Safety categorization for nuclear facility steel structures and structural elements shall be the responsibility of the owner and shall be identified in the contract documents.

Specifically excluded from the Nuclear Specification are the pressure retaining components; for example, pressure vessels, valves, pumps and piping.

In the design of members and connections of seismic force resisting systems, the AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341), hereafter referred to as the Seismic Provisions, in general, are not applicable. However, the detailing requirements of Sections A3 and D2 of the Seismic Provisions shall be appropriately considered when designing for inelastic behavior.

The sponsors of any structural system or construction within the scope of the Nuclear Specification, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by the Nuclear Specification, shall have the right to present the data on which their design is based to the authority having jurisdiction (AHJ) for review and approval.

User Note: For the design of structural members other than hollow structural sections (HSS) that are cold-formed to shapes with elements not more than 1 in. (25 mm) in thickness, the use of provisions of the AISI North American Specification for the Design of Cold-Formed Steel Structural Members is recommended, incorporating the loads and load combinations delineated in Section NB2 of the Nuclear Specification.

NA2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Add the following:
American Institute of Steel Construction (AISC)
ANSI/AISC 360-10 Specification for Structural Steel Buildings

Delete the following:
American Institute of Steel Construction (AISC)
ANSI/AISC N690-2006 Specification for Safety-Related Steel Structures for Nuclear Facilities

Add the following:
American Iron and Steel Institute (AISI)
North American Specification for the Design of Cold-Formed Steel Structural Members, 2007, including Supplement No. 1
American Society of Mechanical Engineers (ASME)

American Institute of Steel Construction
American Society of Civil Engineers (ASCE)
ANSI/ASCE 8-02  Specification for the Design of Cold-Formed Stainless Steel Structural Members

ASTM International (ASTM)
A20/A20M-09  Standard Specification for General Requirements for Steel Plates for Pressure Vessels
A27/A27M-08 Standard Specification for Steel Castings, Carbon, for General Application
A106/A106M-08  Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service
A148/A148M-08  Standard Specification for Steel Castings, High Strength, for Structural Purposes
A217/A217M-08  Standard Specification for Steel Castings, Martensitic Stainless and Alloy, for Pressure-Containing Parts, Suitable for High-Temperature Service
A240/A240M-09  Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications
A276-08a  Standard Specification for Stainless Steel Bars and Shapes
A312/A312M-09  Standard Specification for Seamless, Welded, and Heavily Cold Worked Austenitic Stainless Steel Pipes
A320/A320M-08  Standard Specification for Alloy-Steel Bolting Materials for Low-Temperature Service
A479/A479M-08  Standard Specification for Stainless Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels
A516/A516M-06  Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service
A540/A540M-06  Standard Specification for Alloy-Steel Bolting Materials for Special Applications
A578/A578M-07  Standard Specification for Straight-Beam Ultrasonic Examination of Rolled Steel Plates for Special Applications
A666-03  Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
In addition to satisfying the appropriate ASTM Standards, the specification of the material of those structures or structural components that are subject to suddenly applied dynamic loads (for example, jet shields, pipe whip restraint, or pipe whip impact barriers) shall be supplemented by the requirement that the material be subjected to Charpy V-notch (CVN) impact tests, using the procedures described in ASTM A20/A20M. The CVN impact test shall be conducted at a temperature lower than or equal to 30 °F (17 °C) below the lowest anticipated service temperature of the structural component being evaluated. The acceptance criteria shall be that the
material withstand not less than the energy values (average of three specimens value and individual specimen value) indicated in Table NA3.1, in addition to satisfying the appropriate ASTM Standard.

Certified material test reports (CMTR) or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material meets the ASTM specification and the CVN requirements of Table NA3.1.

In lieu of the above, the material supplier or fabricator shall, if approved by the owner and the AHJ, provide a certificate of compliance stating that the material furnished has been tested and conforms to the ASTM specification and the CVN requirements of Table NA3.1.

1a. ASTM Designations

Modify this section as follows:

(3) Pipe
   Add the following:
   ASTM A106/A106M
   ASTM A312/A312M

(4) Plates
   Add the following:
   ASTM A167
   ASTM A240/A240M
   ASTM A515/A515M
   ASTM A516/A516M

(5) Bars
   Add the following:

<table>
<thead>
<tr>
<th>Specified Minimum Yield Stress</th>
<th>Charpy V-Notch Energy Value</th>
<th>Charpy V-Notch Energy Value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average of Three Specimens, Minimum</td>
<td>One Individual Specimen, Minimum</td>
</tr>
<tr>
<td>Equal to or less than 36 ksi (250 MPa)</td>
<td>15 ft-lb (21 J)</td>
<td>10 ft-lb (14 J)</td>
</tr>
<tr>
<td>Greater than 36 ksi (250 MPa), less than 44 ksi (300 MPa)</td>
<td>20 ft-lb (27 J)</td>
<td>15 ft-lb (21 J)</td>
</tr>
<tr>
<td>Equal to or greater than 44 ksi (300 MPa)</td>
<td>30 ft-lb (41 J)</td>
<td>25 ft-lb (34 J)</td>
</tr>
</tbody>
</table>

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
(6) Sheets

*Add the following:*

- ASTM A666
- ASTM A1008/A1008M

The strength of stainless steel members, assemblies and connections shall be determined in accordance with the requirements in Section 3, 4 and 5 of ANSI/ASCE 8.

**User Note:** Types 301, 301L, 301LN, 302 and 302B of ASTM A167 or ASTM A666, and martensitic stainless steel grades of ASTM A276 should not be used in welded applications.

1b. Unidentified Steel

*Replace section with the following:*

Use of unidentified steel is prohibited.

1c. Rolled Heavy Shapes

*Add the following:*

The project specification covering material for structural components which, as a result of proposed welding procedures, design details, etc., are susceptible to lamellar tearing shall, as determined by the *engineer of record*, include the requirement that the material shall be either ultrasonically examined in accordance with ASTM A578/A578M, Level C or tested in tension in the through-thickness direction (\(z\)-direction). The resulting percentage reduction in area in the \(z\)-direction shall not be less than 90% of that in the direction of material rolling.

**User Note:** In determining the need for pre-fabrication inspection and the inspection acceptance level, the engineer should consider the geometry of the joint, the material type and grade, the anticipated quality of the material, and other experience factors. See Chapter NN. Lamellar tearing is generally caused by the contraction of large metal deposits with high joint restraint; lamellar tears seldom result when weld sizes are less than \(\frac{3}{4}\) in.

1d. Built-Up Heavy Shapes

*Add the following:*

The project specification covering material for structural components which, as a result of proposed welding procedures, design details, etc., are susceptible to lamellar tearing shall, as determined by the engineer of record, include the requirement that the material shall be either ultrasonically examined in accordance with ASTM A578/
A578M, Level C, or tested in tension in the through-thickness direction (z-direction). The resulting percentage reduction in area in the z-direction shall not be less than 90% of that in the direction of material rolling.

**User Note:** Welded joint configurations causing significant through-thickness tensile stress during fabrication, erection and/or service on plate elements of built-up heavy shapes should be avoided. However, if this type of construction is used, the designer should consider one or several of the following factors that may reduce the susceptibility of the joint to experience lamellar tearing:

(a) Reduce the volume of weld metal to the extent practical
(b) Select materials that are resistant to lamellar tearing
(c) Perform through thickness tension testing in accordance with ASTM A770/A770M-03 (2007), *Standard Specification for Through-Thickness Tension Testing of Steel Plates for Special Applications*, for plates (or similar requirements for shapes)
(d) Conduct ultrasonic examination in accordance with ASTM A577/A577M-07 (2007), *Standard Specification for Ultrasonic Angle-Beam Examination of Steel Plates or A578/A578M-07, Standard Specification for Straight-Beam Ultrasonic Examination of Plain and Clad Steel Plated for Special Applications*, of the base material directly underneath the weld after completion of the welding
(e) Use a weld metal inlay or overlay with UT examination after the inlay or overlay but prior to making the welded joint

2. **Steel Castings and Forgings**

*Replace section with the following:*

Steel castings shall conform to ASTM A27/A27M, ASTM A148/A148M, ASTM A216/A216M, or to ASTM A217/A217M. Steel forgings shall conform to ASTM A668/A668M. CMTR or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material meets the ASTM specification.

In lieu of the above, the material supplier or fabricator shall, if approved by the owner and the AHJ, provide a certificate of compliance stating that the steel furnished has been tested and conforms to the ASTM specification.

3. **Bolts, Washers and Nuts**

(1) **Bolts:**

*Add the following:*

ASTM A320/A320M
ASTM A540/A540M
ASTM A564/A564M

American Institute of Steel Construction*
Add the following:

CMTR or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material meets the ASTM specification.

In lieu of the above, the material supplier or fabricator shall, if approved by the owner and the AHJ, provide a certificate of compliance stating that the steel furnished has been tested and conforms to the ASTM specification.

4. Anchor Rods and Threaded Rods

Add the following:

CMTR or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material meets the ASTM specification.

In lieu of the above, the material supplier or fabricator shall, if approved by the owner and the AHJ, provide a certificate of compliance stating that the steel furnished has been tested and conforms to the ASTM specification.

5. Consumables for Welding

Replace section with the following:

Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

AWS A5.1/ A5.1M    AWS A5.20/ A5.20M
AWS A5.4/ A5.4M    AWS A5.23/ A5.23M
AWS A5.5/ A5.5M    AWS A5.25/ A5.25M
AWS A5.9/ A5.9M    AWS A5.26/ A5.26M
AWS A5.17/ A5.17M  AWS A5.28/ A5.28M
AWS A5.18/ A5.18M  AWS A5.29/ A5.29M
AWS A5.32/ A5.32M

Filler material and fluxes that are suitable for the intended application shall be selected. CVN requirements are provided in Section NJ2.6.

CMTR or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material meets the AWS specification.

In lieu of the above, the material supplier or fabricator shall, if approved by the owner and the AHJ, provide a certificate of compliance stating that the material furnished has been tested and conforms to the AWS specification.

6. Headed Stud Anchors

Replace section with the following:

Steel headed stud anchors shall conform to the requirements of Structural Welding Code—Steel, AWS D1.1/D1.1M.
User Note: Studs are made from cold drawn bar conforming to the requirements of ASTM A108, *Standard Specification for Steel Bars, Carbon, Cold-Finished*, standard quality grades 1010 through 1020, inclusive, either semi-killed or killed aluminum or silicon deoxidation.

CMTR or certified reports of tests made by the fabricator or a testing laboratory shall verify that the anchor material and base welds meet the applicable ASTM and AWS specifications.

In lieu of the above, the material supplier or fabricator shall, if approved by the owner and the AHJ, provide a certificate of compliance stating that the steel furnished has been tested and conforms to the ASTM specification.

### NA4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

*Replace section with the following:*

In addition to meeting the provisions of the *Code of Standard Practice*, Section 3, the structural drawings and specifications shall meet the following requirements:

Plans for structural elements shall indicate material, special fabrication and erection requirements, notation of working points for fabrication, and offset dimensions. Members with cyclic loads shall be so indicated as well as the number of cycles, when applicable. The plans for the structural elements shall identify those elements or systems that are deemed safety-related by the *engineer of record*.

The construction specification shall include:

1. Appropriate code references
2. Material specifications
3. Material shipping, handling and storage requirements
4. Surface preparation and protective coating requirements
5. Requirements for fabrication and/or erection
6. Welding and bolting requirements
7. Tests and inspection requirements
8. Requirements for shop drawings
9. Documentation and retention of records

*Add the following section:*

### NA5. QUALITY ASSURANCE

A *quality assurance* program covering *safety-related* steel structures shall be developed prior to design or construction, as applicable. The general requirements and guidelines for establishing and executing the quality assurance program during the design and construction phases of nuclear facilities are those established by Title 10 of the *Code of Federal Regulations*, Part 50 (10CFR50), Appendix B, for Nuclear Power Stations, and as outlined in Chapter NN of the Nuclear Specification.
Calculations pertinent to the design shall be documented and shall include a statement of the applicable design criteria. Calculations shall be performed in accordance with ASME NQA-1, Requirement 3, “Design Control,” or other applicable standards approved by the AHJ. Activities involving specification, design, calculations, drawings, fabrication and erection are subject to quality assurance requirements. Computer programs used in analysis and design shall likewise be covered by a quality assurance program, as provided by ASME NQA-1, Subpart 2.7, “Quality Assurance Requirements for Computer Software for Nuclear Facility Applications.”

User Note: 10CFR50 Appendix B provides regulations for quality assurance (QA) and quality control (QC). The requirements of Chapter NN are aimed to assist the user in developing a QA/QC program that will satisfy the regulations.
Modify Chapter B of the Specification as follows.

Replace preamble with the following:

This chapter addresses general requirements for the analysis and design of steel structures applicable to all chapters of the Nuclear Specification.

NB2. LOADS AND LOAD COMBINATIONS

Replace section with the following:

1. Normal Loads

Normal loads are those loads that are encountered during normal plant start-up, operation and shutdown, and include:

\[ D = \text{dead loads due to the weight of the structural elements, fixed-position equipment, and other permanent appurtenant items; weight of crane trolley and bridge} \]

\[ C = \text{rated capacity of crane (shall include the maximum wheel loads of the crane and the vertical, lateral and longitudinal forces induced by the moving crane)} \]

\[ L = \text{live load due to occupancy and moveable equipment, including impact} \]

\[ L_r = \text{roof live load} \]

\[ R = \text{rain load} \]

\[ R_o = \text{pipe reactions during normal operating, start-up or shutdown conditions, based on the most critical transient or steady-state condition} \]

\[ S = \text{snow load as stipulated in Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7) for Risk Category IV facilities} \]

\[ T_o = \text{thermal effects and loads during normal operating, start-up or shutdown conditions, based on the most critical transient or steady-state condition} \]

2. Severe Environmental Loads

Severe environmental loads are those loads that may be encountered infrequently during the service life, and include:

\[ E_o = \text{where required as part of the design basis, loads generated by the operating basis earthquake (OBE) as defined in the Nuclear Regulatory Commission document, “Earthquake Engineering Criteria for Nuclear Power Plants,” Appendix S, 10 CFR, Part 50, or as specified by the authority having jurisdiction (AHJ)} \]

\[ W = \text{wind load as stipulated in ASCE/SEI 7 for Risk Category IV facilities, or as specified by the AHJ} \]
3. **Extreme Environmental Loads**

Extreme environmental loads are those loads that are highly improbable but are used as a design basis, and include:

\[ E_s = \text{loads generated by the safe shutdown, or design basis earthquake, as defined in the Nuclear Regulatory Commission document, “Earthquake Engineering Criteria for Nuclear Power Plants,” Appendix S, 10 CFR, Part 50, or as specified by the AHJ} \]

\[ W_t = \text{loads generated by the specified design (basis) tornado, including wind pressures, pressure differentials, and tornado-borne missiles, as defined in U.S. Nuclear Regulatory Commission Standard Review Plan 3.3.2 (NUREG-0800) or as specified by the AHJ} \]

4. **Abnormal Loads**

Abnormal loads are those loads generated by a postulated high-energy pipe break accident used as a design basis, and include:

\[ P_a = \text{maximum differential pressure load generated by the postulated accident} \]

\[ R_a = \text{pipe and equipment reactions generated by the postulated accident, including } R_o \]

\[ T_a = \text{thermal loads generated by the postulated accident, including } T_o \]

\[ Y_j = \text{jet impingement load generated by the postulated accident} \]

\[ Y_{m_b} = \text{missile impact load, such as pipe whip generated by or during the postulated accident} \]

\[ Y_r = \text{loads on the structure generated by the reaction of the broken high-energy pipe during the postulated accident} \]

5. **Load and Resistance Factor Design (LRFD)**

The design strength, \( R_n \), of each structural component shall be equal to or greater than the required strength, \( R_u \), determined from the appropriate critical combinations of the loads. The most critical structural effect may occur when one or more loads are not acting. The following load combinations shall be investigated:

5a. **Normal Load Combinations**

\[ 1.4 (D + R_o) + T_o + C \quad \text{(NB2-1)} \]

\[ 1.2 (D + R_o) + 1.6 L + 1.4 \ C + 0.5 (L_r \text{ or } S \text{ or } R) + 1.2T_o \quad \text{(NB2-2)} \]

\[ 1.2 (D + R_o) + 1.6 (L_r \text{ or } S \text{ or } R) + 0.8 L + 1.4 \ C + 1.2T_o \quad \text{(NB2-3)} \]

5b. **Severe Environmental Load Combinations**

\[ 1.2 (D + R_o) + W + 0.8 L + C + 0.5 (L_r \text{ or } S \text{ or } R) + T_o \quad \text{(NB2-4)} \]

\[ 1.2 (D + R_o) + 1.6 E_o + 0.8 L + C + 0.2 (L_r \text{ or } S \text{ or } R) + T_o \quad \text{(NB2-5)} \]

5c. **Extreme Environmental and Abnormal Load Combinations**

\[ D + 0.8 L + C + T_o + R_o + E_s \quad \text{(NB2-6)} \]

\[ D + 0.8 L + T_o + R_o + W_t \quad \text{(NB2-7)} \]
5d. Other Considerations

(1) In applying $T_o$ and $T_o$, the thermal gradient and structural restraint effects shall be considered as appropriate.

(2) Where the structural effect of differential settlement is significant, it shall be included with the dead load.

(3) Where required, loads due to fluids with well-defined pressures shall be treated as dead loads, and loads due to lateral earth pressure, ground water pressure, or pressure of bulk materials shall be treated as live loads.

(4) If the dead load acts to stabilize the structure against the destabilizing effects of lateral force or uplift, the load factor on dead load shall be 0.90 of the assigned factor, and that on other gravity loads ($L, L_r, S, C$) shall be zero.

(5) If the OBE is not part of the design basis, Load Combination NB2-5 need not be evaluated.

(6) In Load Combinations NB2-8 and NB2-9, the maximum values of $P_o, R_o, T_o, Y_r, Y_j$ and $Y_m$, and including an appropriate dynamic load factor, shall be used unless a time-history analysis is performed to justify otherwise. In Load Combination NB2-9, the required strength criteria shall first be satisfied without $Y_r, Y_j$ and $Y_m$. In Load Combinations NB2-7 through NB2-9, when considering concentrated loads $Y_f, Y_r$ and $Y_m$ or tornado-borne missiles, local section strength may be exceeded, as per Section NB3.14, provided that there is no loss of function of any safety-related system.

(7) In addition to the abnormal loads, hydrodynamic loads resulting from a loss of coolant accident (LOCA) and/or safety relief valve actuation shall be appropriately considered for steel structure components subjected to these loads. Any fluid structure interaction associated with these hydrodynamic loads and those from the postulated seismic loads shall be taken into account.

(8) In Load Combination NB2-6, the load $C$ is permitted to be waived, provided it can be demonstrated that the probability of $E_s$ and $C$ occurring at the same time is less than $1 \times 10^{-6}$.

6. Allowable Strength Design (ASD)

The allowable strength, $R_o/\Omega$, of each structural component shall be equal to or greater than the required strength, $R_o$, determined from the appropriate critical combinations of the loads. The most critical structural effects may occur when one or more loads are not acting. The following load combinations shall be investigated:

6a. Normal Load Combinations

\[
D + 0.8L + C + 1.2P_o + R_o + T_o \quad \text{(NB2-8)}
\]

\[
D + 0.8L + (P_o + R_o + T_o) + (Y_r + Y_j + Y_m) + 0.7E_s \quad \text{(NB2-9)}
\]
6b. Severe Environmental Load Combinations

\[ D + R_o + 0.6W + 0.75L + C + 0.75 \ (L_r \text{ or } S \text{ or } R) + T_o \]  
(NB2-13) 

\[ D + R_o + E_o + 0.75L + C + 0.75 \ (L_r \text{ or } S \text{ or } R) + T_o \]  
(NB2-14) 

6c. Extreme Environmental and Abnormal Load Combinations

\[ D + L + C + R_o + T_o + E_s \]  
(NB2-15) 

\[ D + L + R_o + T_o + W_t \]  
(NB2-16) 

\[ D + L + C + P_a + R_a + T_a \]  
(NB2-17) 

\[ D + L + P_a + R_a + T_a + Y_r + Y_j + Y_m + 0.7E_s \]  
(NB2-18) 

6d. Other Considerations

1. In applying \( T_o \) and \( T_a \), the thermal gradient and structural restraint effects shall be considered as appropriate.

2. Where the structural effect of differential settlement is significant, it shall be included with the dead load.

3. Where required, loads due to fluids with well-defined pressures shall be treated as dead loads, and loads due to lateral earth pressure, ground water pressure, or pressure of bulk materials shall be treated as live loads.

4. If the dead load acts to stabilize the structure against the destabilizing effects of lateral force or uplift, the load factor on dead load shall be 0.60 and other gravity loads \( (L, L_r, S, C) \) shall be assumed to equal zero.

5. If the OBE is not part of the design basis, Load Combination NB2-14 need not be evaluated.

6. In Load Combinations NB2-17 and NB2-18, the maximum values of \( P_a, R_o, T_o, Y_r, Y_j \) and \( Y_m \), including an appropriate dynamic load factor, shall be used unless a time-history analysis is performed to justify otherwise. In Load Combination NB2-18, the required strength criteria shall be first satisfied without \( Y_j, Y_r \) and \( Y_m \). In Load Combinations NB2-16 through NB2-18, when considering concentrated loads \( Y_j, Y_r \) and \( Y_m \) or tornado-borne missiles, local section strength may be exceeded as per Section NB3.14, provided that there is no loss of function of any safety-related system.

7. In addition to the abnormal loads, hydrodynamic loads resulting from LOCA and/or safety relief valve actuation shall be appropriately considered for steel structure components subjected to these loads. Any fluid structure interaction associated with these hydrodynamic loads and those from the postulated seismic loads shall be taken into account.

8. For Load Combinations NB2-15 through NB2-18, it is permitted to increase the allowable strength by 1.6. However, this increase shall be limited to 1.5 for members or fasteners in axial tension or in shear.

9. In Load Combination NB2-15, the load \( C \) is permitted to be waived, provided it can be demonstrated that the probability of \( E_s \) and \( C \) occurring at the same time is less than \( 1 \times 10^{-6} \).


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NB3. DESIGN BASIS

*Add the following:*

Buildings and other structures designed by the Nuclear Specification shall be designed using the provisions of either Section NB2.5 (LRFD) or Section NB2.6 (ASD) exclusively throughout the structure.

1. **Required Strength**

*Replace section with the following:*

The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations stipulated in Section NB2. Design by elastic, inelastic or plastic analysis is permitted. Provisions for inelastic and plastic analysis are as stipulated in Appendix N1, Design by Inelastic Analysis.

The yield stress and modulus of elasticity of steel shall be investigated and reduced, as appropriate, for temperatures in excess of 250 °F (121 °C).

**User Note:** Values for the reduction in yield stress and modulus of elasticity of structural steels exposed to elevated temperatures can be found in the Structural Alloys Handbook, published by Battelle, Columbus, OH, and in the ASME Boiler and Pressure Vessel Code, Section II, Part D, Material Properties. Sustained temperature above 700 °F (370 °C) may subject the material to creep rupture effects which need to be considered in the design. Properties for fire conditions of commonly used structural steels are tabulated in Appendix N4, Table NA-4.2.1.

4. **Design for Strength Using Allowable Strength Design (ASD)**

*Add the following:*

It is permitted to multiply the allowable strength by the coefficients stipulated in Section NB2.6d(8).

9. **Design for Serviceability**

*Add the following:*

The effect of elevated temperature on stiffness shall be considered, where appropriate, in calculating structural deformation under operating conditions.

*Add the following section:*

15. **Design Based on Ductility and Local Effects**

In Load Combinations NB2-7 through NB2-9 of Section NB2.5, and in Load Combinations NB2-16 through NB2-18 of Section NB2.6, it is permitted to determine the load effects for *impactive* or *impulsive forces* using inelastic analysis with limits on ductility factors, \( \mu \) (defined as the ratio of permitted strain or deformation to the strain

---

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TABLE NB3.1
Ductility Factor, \( \mu \), for Design of Structural Components for Impactive or Impulsive Forces

<table>
<thead>
<tr>
<th>Description of Element</th>
<th>Ductility Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural steel tension member</td>
<td>( \mu \leq 0.25 \frac{\varepsilon_m}{\varepsilon_y} \leq 0.1/\varepsilon_y ), not to exceed 10 ([b])</td>
</tr>
<tr>
<td>Structural steel flexural members</td>
<td></td>
</tr>
<tr>
<td>Open sections (W, S, WT, etc)</td>
<td>( \mu \leq 10 )</td>
</tr>
<tr>
<td>Closed sections (pipe, box, etc.)</td>
<td>( \mu \leq 20 )</td>
</tr>
<tr>
<td>Members where shear governs design</td>
<td>( \mu \leq 5 )</td>
</tr>
<tr>
<td>Structural steel columns</td>
<td>( \mu = 0.225/(F_y/F_e) \leq \varepsilon_{st}/\varepsilon_y ) not to exceed 10 ([b])</td>
</tr>
</tbody>
</table>

\([a]\) \( \varepsilon_m = \) strain corresponding to elongation at failure (rupture); \( \varepsilon_y = \) strain corresponding to yield stress

\([b]\) \( F_e = \pi^2E/(KL/r)^2; \varepsilon_{st} = \) strain corresponding to the onset of strain hardening

or deformation at yield), equal to one-half the values at the onset of plastic instability, but not to exceed the values given in Table NB3.1. The limiting width-to-thickness ratios for compression elements in members subject to flexure or compression shall not exceed \( \lambda_r \) as given in Table NB3.2. Members in flexure only or combined flexure and compression shall conform to the lateral bracing requirements of Specification Appendix 1, Section 1.2.3.

In designing for impactive and impulsive loads, it is permitted to increase the yield stress used in the determination of nominal strength, \( R_n \). The increase in yield stress shall be determined from supporting experimental data. In the absence of such data, it is permitted to increase the specified yield stress by 10%. Impactive and impulsive loads shall be considered concurrent with other loads in determining the required strength of structural elements.

Areas local to missile and jet impact may be evaluated by means of empirical penetration formulas and no evaluation of local response is required, provided that overall structural stability is assured.


American Institute of Steel Construction
### TABLE NB3.2
Limiting Width-to-Thickness Ratios for Compression Elements per Section NB3.15

<table>
<thead>
<tr>
<th>Description of Element</th>
<th>Width-to-Thickness Ratio</th>
<th>Limiting Width-to-Thickness Ratio $\lambda_r$</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Unstiffened Elements</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flanges of rolled or built-up I-shaped sections, channels and tees</td>
<td>$b/t$</td>
<td>$0.30 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>Legs of single angles or double angle members with separators</td>
<td>$b/t$</td>
<td>$0.30 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>Outstanding legs of pairs of angles in continuous contact</td>
<td>$b/t$</td>
<td>$0.30 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>Flanges of H-pile sections</td>
<td>$b/t$</td>
<td>$0.45 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>Stems of tees</td>
<td>$d/t$</td>
<td>$0.30 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td><strong>Stiffened Elements</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Walls of rectangular HSS</td>
<td>$b/t$</td>
<td>$0.55 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>Flanges of boxed I-shaped sections and built-up box sections</td>
<td>$b/t$</td>
<td>$0.55 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces</td>
<td>$b/t$</td>
<td>$0.55 \sqrt{E/F_y}$</td>
<td></td>
</tr>
</tbody>
</table>
### TABLE NB3.2 (continued)
Limiting Width-to-Thickness Ratios for Compression Elements per Section NB3.15

<table>
<thead>
<tr>
<th>Description of Element</th>
<th>Width-to-Thickness Ratio</th>
<th>Limiting Width-to-Thickness Ratio $\lambda_r$</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Stiffened Elements</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| Webs of rolled or built-up I-shaped sections used for beams or columns | $h/t_w$ | For $C_a \leq 0.125$ $2.45 \sqrt{E/F_y} (1 - 0.93C_a)$  
For $C_a > 0.125$ $0.77 \sqrt{E/F_y} (2.93 - C_a)$ $\geq 1.49 \sqrt{E/F_y}$  
where $C_a = \frac{P_y}{\phi C a_y}$ (LRFD)  
$C_a = \frac{\Omega \phi a_y}{P_y}$ (ASD) | ![Example](image1.png) |
| Side plates of boxed I-shaped sections used as beams or columns | $h/t$ | ![Example](image2.png) |
| Webs of built-up box sections used as beams or columns | $h/t$ | ![Example](image3.png) |
| Webs of rolled or built-up I-shaped sections used as diagonal braces | $h/t_w$ | $1.49 \sqrt{E/F_y}$ | ![Example](image4.png) |
| Webs of H-Pile sections | $h/t_w$ | $0.94 \sqrt{E/F_y}$ | ![Example](image5.png) |
| Walls of round HSS | $D/t$ | $0.038E / F_y$ | ![Example](image6.png) |
### TABLE NB3.2 (continued)
Limiting Width-to-Thickness Ratios for Compression Elements per Section NB3.15

<table>
<thead>
<tr>
<th>Description of Element</th>
<th>Width-to-Thickness Ratio</th>
<th>Limiting Width-to-Thickness Ratio $\lambda_r$</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite Elements</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| Walls of rectangular filled composite members | $b/t$ | $1.40 \sqrt{E/F_y}$ | ![Diagram](b/t)
| Walls of round filled composite members | $D/t$ | $0.076E/F_y$ | ![Diagram](D/t) |

[a] For tee shaped compression members, the limiting width-to-thickness ratio for highly ductile members for the stem of the tee can be increased to $0.38 \sqrt{E/F_y}$ if either of the following conditions are satisfied:

1. Buckling of the compression member occurs about the plane of the stem.
2. The axial compression load is transferred at end connections to only the outside face of the flange of the tee resulting in an eccentric connection that reduces the compression stresses at the tip of the stem.

[b] The limiting width-to-thickness ratio of flanges of boxed I-shaped sections and built-up box sections of columns in SMF systems shall not exceed $0.6 \sqrt{E/F_y}$.

### NB5. FABRICATION AND ERECTION

Replace section with the following:

Shop drawings, fabrication, shop painting, erection and quality control shall meet the requirements in Chapter NM, Fabrication and Erection.

### NB6. QUALITY CONTROL AND QUALITY ASSURANCE

Replace section with the following:

Quality control and quality assurance activities shall satisfy the requirements stipulated in Section NA5, Quality Assurance, and Chapter NN, Quality Control and Quality Assurance.

### NB7. EVALUATION OF EXISTING STRUCTURES

Replace section with the following:

Provisions for the evaluation of existing structures are presented in Appendix N5, Evaluation of Existing Structures.

*Specification for Safety-Related Steel Structures for Nuclear Facilities, January 31, 2012*

**AMERICAN INSTITUTE OF STEEL CONSTRUCTION**
CHAPTER NC
DESIGN FOR STABILITY

Modify Chapter C of the Specification as follows.

Add the following item to the list of (5) five in the first paragraph of C1:

(6) and the effects of elevated temperatures.
CHAPTER ND
DESIGN OF MEMBERS FOR TENSION

No changes to Chapter D of the Specification.
CHAPTER NE
DESIGN OF MEMBERS FOR COMPRESSION

No changes to Chapter E of the Specification.
CHAPTER NF
DESIGN OF MEMBERS FOR FLEXURE

*No changes to Chapter F of the Specification.*
CHAPTER NG
DESIGN OF MEMBERS FOR SHEAR

No changes to Chapter G of the Specification.
CHAPTER NH

DESIGN OF MEMBERS FOR
COMBINED FORCES AND TORSION

No changes to Chapter H of the Specification.
CHAPTER NI
DESIGN OF COMPOSITE MEMBERS

Modify Chapter I of the Specification as follows.

Replace “ACI 318” with “ACI 349”

In Section I1.1(1), replace “ACI 318, Sections 7.8.2 and 10.13” with “ACI 349, Sections 7.8.2 and 10.16.”

Replace User Note of Section I8.3b with the following:

User Note: Supplemental confining reinforcement is recommended around the anchors for steel headed stud anchors subjected to tension or interaction of shear and tension to avoid edge effects or effects from closely spaced anchors. See the Commentary and ACI 349, Appendix D for guidelines.
**CHAPTER NJ**

**DESIGN OF CONNECTIONS**

*Modify Chapter J of the Specification as follows.*

**NJ1. GENERAL PROVISIONS**

*Modify section as follows:*

*Replace Section J1.9 with the following:*

**9. Rivets**

Rivets shall not be used in *safety-related* nuclear facilities.

**10. Limitations on Bolted and Welded Connections**

*Replace section with the following:*

Pretensioned high-strength bolts (see *Specification* Table J3.1) or welds shall be used for the following connections:

1. All column splices
2. Connections of beams and girders to columns in which the bracing of columns is dependent
3. Roof-truss splices and connections of trusses to columns, column splices, column bracing knee braces, and crane supports
4. Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress

**User Note:** For vibrating machinery supports and other situations where high-cycle fatigue may be a design concern, the use of slip-critical joints represents good design practice. However, properly designed welds may be used. See Appendix N3 for design of joints subject to high cycle fatigue.

5. Any other connections stipulated on the design documents

In other cases, connections are permitted to be made with ASTM A307 bolts or snug-tight high-strength bolts.

Bolted connections for members that are part of the seismic force resisting system and/or are subjected to dynamic loads shall be configured such that a ductile limit state in either the member or the connection controls the design.
NJ2. WELDS

Modify section as follows:

2b. Limitations

Replace introductory phrase that begins with “Fillet weld terminations are permitted…” with the following:

Fillet weld terminations shall comply with the following limitations:

Replace User Note in this section with the following:

**User Note:** Fillet welded items that do not fall into the above-listed categories should terminate short of the material end. In such cases, the welded connection design should assume that welds will be terminated short of the end of the joint.

6. Filler Metal Requirements

Replace second paragraph with the following:

Filler metal with a minimum specified Charpy V-notch (CVN) toughness of 20 ft-lb (27 J) at 40 °F (4 °C) or lower shall be used in the following joints:

(1) Complete-joint-penetration groove welded T- and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the nominal strength and resistance factor or safety factor as applicable for a partial-joint-penetration groove weld.

(2) Complete-joint-penetration groove welded splices subject to tension normal to the effective area in heavy sections as defined in Sections A3.1c and A3.1d.

Where welds are designated critical by the **engineer of record**, they shall be made with a filler metal capable of providing a minimum Charpy V-notch (CVN) toughness of 20 ft-lb (27 J) at −20 °F (−29 °C) as determined by the appropriate AWS classification test method or manufacturer certification, and 40 ft-lb (54 J) at 70 °F (21 °C) as determined by the appropriate AWS classification test method or manufacturer certification, when the steel frame is normally enclosed and maintained at a temperature of 50 °F (10 °C) or higher. For structures with service temperatures lower than 50 °F (10 °C), instead of 70 °F (21 °C) the qualification temperature shall be 20 °F (11 °C) above the lowest anticipated service temperature, or at a lower temperature. SMAW electrodes classified in AWS A5.1 as E7018 or E7018-X, SMAW electrodes classified in AWS A5.5 as E7018-C3L or E8018-C3, and GMAW solid electrodes are exempted from production lot testing when the CVN toughness of the electrode equals or exceeds 20 ft-lb (27 J) at a temperature not exceeding 20 °F (29 °C) as determined by AWS classification test methods.
NJ3. BOLTS AND THREADED PARTS

Modify section as follows:

8. High-Strength Bolts in Slip-Critical Connections

Add the following:

All faying surfaces shall be prepared as required for Class A or better surfaces.

10. Bearing Strength at Bolt Holes

Replace paragraph (a) with the following:

(a) For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:

\[ R_n = 1.2L_c t F_u \leq 2.4d t F_u \]  

User Note: Equation J3-6 replaces Equations J3-6a and J3-6b in the Specification. Deformation at bolt holes is always a design consideration in nuclear facilities. Paragraphs b and c of Specification Section J3.10 remain unchanged.
CHAPTER NK
DESIGN OF HSS AND BOX MEMBER CONNECTIONS

Replace preamble’s second User Note of the Specification as follows:

**User Note:** See also Chapter J of the *Specification* and as modified by Chapter NJ of the Nuclear Specification for additional requirements for bolting and welding to HSS material.
CHAPTER NL
DESIGN FOR SERVICEABILITY

Modify Chapter L of the Specification as follows.

Replace preamble with the following:

This chapter addresses serviceability design requirements.

NL1. GENERAL PROVISIONS

Replace section with the following:

Serviceability of a nuclear plant structure is a state in which the function of a structure, its maintainability, durability and the ability of safety-related systems and components to perform their intended design function are preserved under various loading conditions. Limiting values of structural behavior for serviceability (for example, maximum deflections or accelerations) shall be chosen by the engineer of record with due regard to the intended safety-related function of the structure. Serviceability shall be evaluated using appropriate load combinations stipulated in Section NB2 and the applicable Appendices.
CHAPTER NM
FABRICATION AND ERECTION

Modify Chapter M of the Specification as follows.

NM1. SHOP AND ERECTION DRAWINGS

Replace section with the following:

Shop and erection drawings are permitted to be prepared in stages. Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted connections.

Unless otherwise noted in the contract documents, a response to a request for information, as defined in Section 4.6 of the Code of Standard Practice, shall constitute design direction and a release for construction.

Shop and erection drawings shall have a means of indicating which parts are safety-related.

NM2. FABRICATION

1. Cambering, Curving and Straightening

Modify section to read as follows:

Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature and straightness. The temperature of heated areas shall not exceed 1,100 °F (593 °C) for A514/A514M and A852/A852M steel nor 1,200 °F (649 °C) for other carbon steels. The temperature of heated areas for ferritic, martensitic or duplex stainless steels shall not exceed 600 °F (316 °C). The temperature of heated areas for austenitic stainless steel shall not exceed 800 °F (433 °C). The temperature of heated areas for precipitation hardening stainless steel shall not exceed the ageing temperature.

2. Thermal Cutting

Modify first paragraph to read as follows:

Thermally cut edges shall meet the requirements of AWS D1.1/D1.1M, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will not be subject to fatigue shall be free of round-bottom gouges greater than ⅛ in. (5 mm) deep and sharp V-shaped notches. Gouges deeper than ⅛ in. (5 mm) and
notches shall be removed by grinding or repaired by welding. Notches or gouges greater than 1/16 in. (5 mm) up to 3/4 in. (10 mm) deep that remain from cutting shall be removed by grinding at a slope of 1 to 2. Notches or gouges 3/8 in. (5 mm) deep or greater shall be repaired only with the approval of the engineer of record. Oxygen gouging is not permitted on quenched and tempered steels.

3. Planing of Edges

*Replace section with the following:*

Planing or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the construction documents or included in a stipulated edge preparation for welding. Planed or finished edges shall not vary by more than 1/8 in. (3 mm) from a true plane.

4. Welded Construction

*Replace section with the following:*

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1/D1.1M and AWS D1.6/D1.6M except as modified in Section J2.

7. Dimensional Tolerances

*Replace section with the following:*

Dimensional tolerances shall be in accordance with *Code of Standard Practice*, Section 6, and as listed below.

For acceptable tolerances not found in the *Code of Standard Practice* or not listed below, the engineer of record shall provide the necessary tolerances.

1. **Holes**

   A variation from the detailed distance of 1/16 in. (2 mm) center-to-center of holes is permissible for members 30 ft (9 m) or less, and 3/8 in. (4 mm) for members over 30 ft (9 m) in length.

   In compression members, erection holes or holes mis-punched or mis-drilled may be left unfilled provided the net area is not less than 0.85 times the gross area. In tension members holes may be left unfilled provided the net area requirements are met. In either condition the unfilled holes may not violate the minimum hole spacing requirements of Section J3.3.

2. **Stiffeners**

   Stiffeners serving as connections shall be located within 1/4 in. (7 mm) of the detailed position. A variation of 1 in. (25 mm) is permissible for the location of other stiffeners except bearing stiffeners which shall be within 1/2 of their thickness from the detailed position.
(3) **Welding**

The fabrication tolerance of welded structural members shall conform to the provisions of AWS D1.1/D1.1M or AWS D1.6/D1.6M, as applicable.

9. **Holes for Anchor Rods**

*Replace section with the following:*

Holes for anchor rods are permitted to be thermally cut in accordance with the provisions of Section NM2.2.

*Add the following new sections:*

12. **Surface Condition**

Procedures for inspection and correcting surface defects in excess of the depth and area limitations of those specified in ASTM A6/A6M or other applicable ASTM specifications shall include the inspection method and acceptance criteria to be used.

13. **Bending**

The minimum bending radius for plates shall not be less than that specified for the bend test in the applicable material standard.

14. **Commercial Grade Dedication**

If not available from a qualified source, the material shall be dedicated for use as specified in Subpart 2.14 of the NQA-1. The engineer of record shall provide the fabricator with the critical material characteristics based on the applicable ASTM or other national standard as necessary for *dedication* of this material.

15. **Identification of Steel**

The fabricator shall be able to demonstrate, by written procedure and by actual practice, a method of material identification meeting the requirements of the contract documents.

The material will be identified in one of the following ways as defined by the required use of the material. The material’s use must be defined by the contract documents. If the contract documents do not define the type of identification required, the identification defined in item (1) below will control.

1. **Material identified by grade and size only.** Material need only be identified in such a manner that the purchaser is assured that the specified grade is used, and this documentation can be obtained throughout the service life of the structure.

2. **Material identified by heat number for the structure only.** Material test reports shall be identifiable to the structure, but need not be to an individual member in the structure, in such a manner that the material test report can be obtained throughout the service life of the structure.
(3) Material identified by heat number for an individual member, but not subparts, fasteners, or weld consumables. Material test reports shall be identifiable to an individual member in the structure in such a manner that the material test report can be obtained throughout the service life of the structure.

(4) Material identified by heat or production lot number to all components of the structure including subparts, fasteners and weld consumables. Material test reports shall be identifiable to an individual member, subpart, fastener or weld consumable in such a manner that the material test report can be obtained throughout the service life of the structure.

NM3. SHOP PAINTING

4. Finished Surfaces

*Replace section with the following:*

Except for stainless steels, machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary. Such rust inhibitive coating shall be approved by the *engineer of record.*

*Add the following User Note:*

**User Note:** Paint (coatings) procurement, application and inspection for a nuclear facility is subject to multiple codes, standards and regulations which may vary substantially from typical fabricator requirements. Contract documents and design specifications should be consulted for specific information.

NM4. ERECTION

2. Stability and Connections

*Replace section with the following:*

The frame of structural steel buildings and composite steel/concrete (SC) structures shall be carried up true and plumb within the limits defined in the *Code of Standard Practice* and/or Contract Documents. Temporary bracing shall be provided in accordance with the requirements of the *Code of Standard Practice* and/or Contract Documents wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. For composite steel/concrete structures, the required bracing must satisfactorily resist impact and hydrostatic loads of fluid concrete during placement of concrete within the structure. Bracing shall be left in place as long as required for safety.

*Add the following new sections:*

7. Tolerances for Cranes

7a. Tolerances for Crane Column Base Lines

*Specification for Safety-Related Steel Structures for Nuclear Facilities, January 31, 2012*

*American Institute of Steel Construction*
Crane column base lines shall be established as parallel lines and the columns centerlines maintained within ⅛ in. (3 mm) of the theoretical distance.

7b. **Tolerances for Crane Runway Girders**

Horizontal sweep in crane runway girders shall not exceed ¼ in. (6 mm) per 50 ft (15 m) length of girder spans. Camber shall not exceed ¼ in. (6 mm) per 50 ft (15 m) of the girder span over that indicated on the design drawings.

7c. **Tolerances for Crane Rails**

Center-to-center distances of crane rails and the straightness of crane rails shall meet the tolerances prescribed by “Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes” (CMAA-70). Vertical misalignment of crane rails measured at centerlines of columns shall meet the tolerances prescribed by CMAA-70. For polar cranes, the above tolerances shall apply except that the CMAA tolerances for crane span shall be applied for crane rail diameter. Crane rails shall be centered on the crane girders wherever possible. For plate girders and wide-flange shapes (i.e., not box beams), in no case shall the real eccentricity be greater than ¼ of the thickness of the web, unless such eccentricity is accounted for in design.
CHAPTER NN
QUALITY CONTROL AND QUALITY ASSURANCE

Replace Chapter N of the Specification with the following:

This chapter addresses minimum requirements for quality control, quality assurance and nondestructive evaluation for safety-related structural steel systems and steel elements of composite members for nuclear facilities.

User Note: This chapter does not address quality control or quality assurance for concrete reinforcing bars, concrete materials or placement of concrete for composite members. This chapter does not address quality control or quality assurance for surface preparation or coatings.

User Note: The inspection of steel (open-web) joists and joist girders, tanks, pressure vessels, cables, cold-formed steel products, or gage metal products is not addressed in this Specification.

User Note: The provisions of this chapter are pertinent to the activities performed by the fabricator, erector and associated parties. Consult Section NA5 for activities related to calculations and design.

The Chapter is organized as follows:

NN2. Fabricator and Erector Quality Assurance Program
NN3. Fabricator and Erector Documents
NN4. Inspection and Nondestructive Evaluation Personnel
NN5. Minimum Requirements for Inspection of Structural Steel Buildings and Structures
NN6. Minimum Requirements for Inspection of Composite Construction
NN7. Nonconforming Material and Workmanship

NN1. GENERAL PROVISIONS

The fabricator and erector shall include both quality control (QC) and quality assurance (QA) as part of their quality plan as specified in this chapter. When required by the authority having jurisdiction (AHJ), applicable building code (ABC), purchaser, owner or engineer of record, an independent party shall provide additional oversight to ensure the fabricator and erector are following their QA program. Nondestructive examination (NDE) shall be performed by an individual, agency or firm approved by the fabricator or erector responsible for quality assurance.
User Note: The producers of materials manufactured in accordance with standard specifications referenced in Section NA3 in this Nuclear Specification, and steel deck manufacturers, are not considered fabricators or erectors.

NN2. FABRICATOR AND ERECTOR QUALITY ASSURANCE PROGRAM

The fabricator and erector shall establish, maintain and document procedures and perform inspections to ensure that their work is performed in accordance with the established \textit{quality assurance} program, the appropriate elements of the standard, this Nuclear Specification, and the construction documents. The quality assurance program shall be developed based on national consensus standards such as ASME standard NQA-1, \textit{Quality Assurance Requirements for Nuclear Facility Applications}, or equivalent.

Material identification procedures shall comply with the requirements of the \textit{Code of Standard Practice} Section 6.1 except that the identification of material deemed safety related will be maintained, retrievable, traceable and transferred to the owner at the time of delivery as defined in Section NM2.15. The procedure will be monitored by the individual responsible for the fabricator’s quality program.

The fabricator’s \textit{quality assurance inspector (QAI)} shall inspect to the approved shop drawings the following as a minimum, as applicable:

(1) Shop welding, high-strength bolting, and details in accordance with Section NN5

(2) Shop cut and finished surfaces, in accordance with Section NM2

(3) Shop heating for straightening, cambering and curving, in accordance with Section NM2.1

(4) Tolerances for shop fabrication, in accordance with Section 6 of the \textit{Code of Standard Practice} and Chapter NM.

The erector’s QAI shall inspect, to the approved erection and installation drawings, the following as a minimum, as applicable:

(1) Field welding, high-strength bolting, and details in accordance with Section NN5

(2) Steel deck and headed steel stud anchor placement and attachment in accordance with Section NN6

(3) Field cut surfaces, in accordance with Section NM2.2

(4) Field heating for straightening, in accordance with Section NM2.1

(5) Tolerances for field erection, in accordance with Section 7.13 of the \textit{Code of Standard Practice} and Chapter NM.
NN3. FABRICATOR AND ERECTOR DOCUMENTS

1. Submittals for Steel Construction

The fabricator or erector shall submit the following documents in electronic or printed form for review by the owner or the engineer of record or their designee, in accordance with Section 4.4 of the Code of Standard Practice, prior to fabrication or erection, as applicable:

(1) Shop drawings, unless shop drawings have been furnished by the owner or the engineer of record

(2) Erection drawings, unless erection drawings have been furnished by the owner or the engineer of record

At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the fabricator are in accordance with the construction documents. At completion of erection, the approved erector shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the erector are in accordance with the construction documents.

2. Available Documents for Steel Construction

The following documents shall be available in electronic or printed form for review and approval, as applicable, by the engineer of record or the engineer of record’s designee prior to fabrication or erection, as applicable, unless otherwise required in the contract documents to be submitted:

(1) For structural steel elements, copies of material test reports in accordance with Section NA3.1.

(2) For steel castings and forgings, copies of material test reports in accordance with Section NA3.2.

(3) For fasteners, copies of manufacturer’s certifications in accordance with Section NA3.3.

(4) For deck fasteners, copies of manufacturer’s product data sheets or catalog data. The data sheets shall describe the product, limitations of use, and recommended or typical installation instructions.

(5) For anchor rods and threaded rods, copies of material test reports in accordance with Section NA3.4.

(6) For welding consumables, copies of manufacturer’s certifications in accordance with Section NA3.5.

(7) For headed stud anchors, copies of manufacturer’s certifications in accordance with Section NA3.6.

(8) Manufacturer’s product data sheets or catalog data for welding filler metals and fluxes to be used. The data sheets shall describe the product, limitations of use,
recommended or typical welding parameters, and storage and exposure requirements, including baking, if applicable.

(9) Welding procedure specifications (WPS).

(10) Procedure qualification records (PQR) for WPS that are not prequalified in accordance with AWS D1.1/D1.1M, AWS D1.6/D1.6M, or AWS D1.3/D1.3M, as applicable.

(11) Welding personnel performance qualification records (WPQR) and continuity records.

(12) Fabricator’s or erector’s written quality assurance manual, as applicable.

(13) Fabricator’s or erector’s QC inspector qualifications, as applicable.

**NN4. INSPECTION AND NONDESTRUCTIVE EVALUATION PERSONNEL**

1. **Quality Control Inspector Qualifications**

Quality control (QC) welding inspectors shall be qualified to the satisfaction of the fabricator’s or erector’s quality assurance (QA) program.

QC bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection as defined in the QA program.

2. **Quality Assurance Inspector Qualifications**

QA welding inspectors shall be qualified to the satisfaction of the fabricator’s or erector’s QA program, the owner’s written requirements, and in accordance with either of the following:

   (a) Welding inspectors (WI) or senior welding inspectors (SWI), as defined in AWS B5.1, *Standard for the Qualification of Welding Inspectors*, except associate welding inspectors (AWI) are permitted to be used under the direct supervision of WI, who are on the premises and available when weld inspection is being conducted, or

   (b) Qualified under the provisions of AWS D1.1/D1.1M, subclause 6.1.4.

QA bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection as defined in the QA program.

3. **NDE Personnel Qualifications**

Nondestructive examination personnel for NDE shall be qualified in accordance with their employer’s written practice, which shall meet the criteria of AWS D1.1/D1.1M *Structural Welding Code—Steel*, subclause 6.14.6, and:

   (1) American Society for Nondestructive Testing (ASNT) SNT-TC-1A, *Recommended Practice for the Qualification and Certification of Nondestructive Testing Personnel*, or
MINIMUM REQUIREMENTS FOR INSPECTION

(2) 

**NN5. MINIMUM REQUIREMENTS FOR INSPECTION OF STRUCTURAL STEEL BUILDINGS AND STRUCTURES**

1. **Quality Control**

QC inspection tasks shall be performed by personnel qualified as defined in Section NN4.1, as applicable, in accordance with Sections NN5.4, NN5.6 and NN5.7.

Tasks in Tables NN5.4-1 through NN5.4-3 and Tables NN5.6-1 through NN5.6-3 listed for QC are those inspections performed by qualified personnel to ensure that the work is performed in accordance with the construction documents.

For QC inspection, the applicable construction documents are the approved shop drawings and the erection drawings, and the applicable referenced specifications, codes and standards.

**User Note:** The personnel performing QC inspection need not refer to the design drawings and project specifications. The *Code of Standard Practice*, Section 4.2(a), requires the transfer of information from the contract documents (design drawings and project specification) into accurate and complete shop and erection drawings, allowing QC inspection to be based upon shop and erection drawings alone.

2. **Quality Assurance**

Quality assurance (QA) inspection of fabricated items shall be made at the fabricator’s plant. The quality assurance inspector (QAI) shall schedule this work to minimize interruption to the work in fabrication.

QA inspection of the erected steel system shall be made at the project site. The QAI shall schedule this work to minimize interruption to the work during erection.

The QAI or qualified personnel identified in the QA program shall review the material test reports and certifications as listed in Section NN3.2 for compliance with the construction documents before the material is shipped from the fabricator’s plant.

QA inspection tasks shall be performed by the QAI, in accordance with Sections NN5.4, NN5.6 and NN5.7.

Tasks in Tables NN5.4-1 through NN5.4-3 and Tables NN5.6-1 through NN5.6-3 listed for QA are those inspections performed by the QAI to ensure that the work is performed in accordance with the construction documents.

For QA inspection, the applicable construction documents are the approved shop drawings and the erection drawings, specifications, and applicable reference codes and standards.
3. **Coordinated Inspection**

Where a task is noted to be performed by both QC and QA, it is permitted to coordinate the inspection function between the personnel qualified for quality control inspection and QAI so that the inspection functions are performed by only one party. Where QA relies upon inspection functions performed by personnel qualified for quality control inspection, the approval of the *engineer of record* and the AHJ is required and the procedure must be stated in the QA program.

4. **Inspection of Welding**

Observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the construction documents. For structural steel, applicable provisions of AWS D1.1/D1.1M, D1.6/D1.6M or D1.3/D1.3M shall apply to all structural steel.

**User Note:** Section J2 of the *Specification* contains exceptions to AWS D1.1/D1.1M.

As a minimum, welding inspection tasks shall be in accordance with Tables NN5.4-1, NN5.4-2 and NN5.4-3. In these tables, the inspection tasks are as follows:

- **O**—Observe these items on a random basis. Operations need not be delayed for performing these inspections.

- **P**—Perform these tasks for each welded joint or member.
### TABLE NN5.4-1
**Inspection Tasks Prior to Welding**

<table>
<thead>
<tr>
<th>Inspection Tasks Prior to Welding</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welding procedure specifications (WPSs) available</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Manufacturer certifications for welding consumables available</td>
<td>N/A</td>
<td>P</td>
</tr>
<tr>
<td>Material identification (type/grade)</td>
<td>N/A</td>
<td>O</td>
</tr>
<tr>
<td>Welder identification system(^1)</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Fit-up of groove welds (including joint geometry)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Joint preparation</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>• Dimensions (alignment, root opening, root face, bevel)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Cleanliness (condition of steel surfaces)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Tacking (tack weld quality and location)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Backing type and fit (if applicable)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Configuration and finish of access holes</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Fit-up of fillet welds</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Dimensions (alignment, gaps at root)</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>• Cleanliness (condition of steel surfaces)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Tacking (tack weld quality and location)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Check welding equipment</td>
<td>P</td>
<td>O</td>
</tr>
</tbody>
</table>

\(^1\) The fabricator or erector, as applicable, shall maintain a system by which a welder who has welded a joint or member can be identified. Stamps, if used, shall be the low-stress type.

N/A = not applicable
### TABLE NN5.4.-2
**Inspection Tasks During Welding**

<table>
<thead>
<tr>
<th>Inspection Tasks During Welding</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Use of qualified welders</td>
<td>N/A</td>
<td>O</td>
</tr>
<tr>
<td>Control and handling of welding consumables</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Packaging</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>• Exposure control</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No welding over cracked tack welds</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Environmental conditions</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Wind speed within limits</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>• Precipitation and temperature</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WPS followed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Settings on welding equipment</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>• Travel speed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Selected welding materials</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Shielding gas type/flow rate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Preheat applied</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Interpass temperature maintained (min./max.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Proper position (F, V, H, OH)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welding techniques</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Interpass and final cleaning</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>• Each pass within profile limitations</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Each pass meets quality requirements</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
TABLE NN5.4-3
Inspection Tasks After Welding

<table>
<thead>
<tr>
<th>Inspection Tasks After Welding</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welds cleaned</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Size, length and location of welds</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Welds meet visual acceptance criteria</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Crack prohibition</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Weld/base-metal fusion</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Crater cross section</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Weld profiles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Weld size</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Undercut</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Porosity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arc strikes</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>k-area&lt;sup&gt;1&lt;/sup&gt;</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Backing removed and weld tabs removed (if required)</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Repair activities</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Document acceptance or rejection of welded joint or member</td>
<td>P</td>
<td>O</td>
</tr>
</tbody>
</table>

<sup>1</sup> When welding of doubler plates, continuity plates or stiffeners has been performed in the k-area, visually inspect the web k-area for cracks within 3 in. (75 mm) of the weld.

5. Nondestructive Examination of Welded Joints

5a. Procedures

Ultrasonic testing (UT), magnetic particle testing (MT), penetrant testing (PT), and radiographic testing (RT), where required, shall be performed by qualified NDE personnel in accordance with AWS D1.1/D1.1M and AWS D1.6/D1.6M, as applicable. Acceptance criteria shall be AWS D1.1/D1.1M and AWS D1.6/D1.6M, as applicable, for statically loaded structures, unless otherwise designated in the design drawings or project specifications.

5b. CJP Groove Weld NDE

UT shall be performed by qualified NDE personnel on complete-joint-penetration (CJP) groove welds subject to transversely applied tension loading in butt, T- and corner joints, in materials ½ in. (8 mm) thick or greater.

5c. Access Hole NDE

Thermally cut surfaces of access holes shall be tested by qualified NDE personal using MT or PT, when the flange thickness exceeds 2 in. (50 mm) for rolled shapes.
or when the web thickness exceeds 2 in. (50 mm) for built-up shapes. Any crack shall be deemed unacceptable regardless of size or location.

User Note: See Section NM2.2.

5d. **Welded Joints Subjected to Fatigue**

When required by Specification Appendix 3, Table A-3.1, welded joints requiring weld soundness to be established by radiographic or ultrasonic inspection shall be tested by qualified NDE personal as prescribed. Reduction in the rate of UT is prohibited.

5e. **Reduction of Rate of Ultrasonic Examination**

The rate of UT is permitted to be reduced if approved by the engineer of record or the AHJ. Where the initial rate for UT is 100%, the NDE rate for an individual welder or welding operator is permitted to be reduced to 25%, provided the rejection rate, the number of welds containing unacceptable defects divided by the number of welds completed, is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 60 completed welds for a job shall be made for such reduction evaluation. For evaluating the reject rate of continuous welds over 3 ft (1 m) in length where the effective throat is 1 in. (25 mm) or less, each 12 in. (300 mm) increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 ft (1 m) in length where the effective throat is greater than 1 in. (25 mm), each 6 in. (150 mm) of length or fraction thereof shall be considered one weld. No reduction in the rate of UT testing for welds subject to impactive or impulsive loads shall be allowed.

5f. **Increase in Rate of Ultrasonic Examination**

For structures in which the initial rate for UT is 10%, the NDE rate for an individual welder or welding operator shall be increased to 100% should the reject rate, the number of welds containing unacceptable defects divided by the number of welds completed, exceed 5% of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds for a job shall be made prior to implementing such an increase. When the reject rate for the welder or welding operator, after a sampling of at least 40 completed welds, has fallen to 5% or less, the rate of UT shall be returned to 10%. For evaluating the reject rate of continuous welds over 3 ft (1 m) in length where the effective throat is 1 in. (25 mm) or less, each 12 in. (300 mm) increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 ft (1 m) in length where the effective throat is greater than 1 in. (25 mm), each 6 in. (150 mm) of length or fraction thereof shall be considered one weld.

5g. **Documentation**

All NDE performed shall be documented. For shop fabrication, the NDE report shall identify the tested weld by piece mark and location in the piece. For field work, the NDE report shall identify the tested weld by location in the structure, piece mark, and location in the piece.


**American Institute of Steel Construction**
6. Inspection of High-Strength Bolting

Observation of bolting operations shall be the primary method used to confirm that the materials, procedures and workmanship incorporated in construction are in conformance with the construction documents and the provisions of the RCSC Specification for Structural Joints Using High-Strength Bolts, hereafter referred to as the RCSC Specification.

(1) For snug-tight joints, pre-installation verification testing as specified in Table NN5.6-1 and monitoring of the installation procedures as specified in Table NN5.6-2 are not applicable. The QAI need not be present during the installation of fasteners in snug-tight joints.

(2) For pretensioned joints and slip-critical joints, when the installer is using the turn-of-nut method with matchmarking techniques, the direct-tension-indicator method, or the twist-off-type tension control bolt method, monitoring of bolt pretensioning procedures shall be as specified in Table NN5.6-2. The QAI need not be present during the installation of fasteners when these methods are used by the installer.

(3) For pretensioned joints and slip-critical joints, when the installer is using the calibrated wrench method or the turn-of-nut method without matchmarking, monitoring of bolt pretensioning procedures shall be as specified in Table NN5.6-2. The QAI shall be engaged in their assigned inspection duties during installation of fasteners when these methods are used by the installer.

As a minimum, bolting inspection tasks shall be in accordance with Tables NN5.6-1, NN5.6-2 and NN5.6-3. In these tables, the inspection tasks are as follows:

O—Observe these items on a random basis. Operations need not be delayed pending these inspections.

P—Perform these tasks for each bolted connection.
### TABLE NN5.6-1
**Inspection Tasks Prior to Bolting**

<table>
<thead>
<tr>
<th>Inspection Tasks Prior to Bolting</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manufacturer’s certifications available for fastener materials</td>
<td>N/A</td>
<td>P</td>
</tr>
<tr>
<td>Fasteners marked in accordance with ASTM requirements</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Proper fasteners selected for the joint detail (grade, type, bolt length if threads are to be excluded from shear plane)</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Proper bolting procedure selected for joint detail</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Pre-installation verification testing by installation personnel observed and documented for fastener assemblies and methods used (Reference RCSC Specification, Section 7 )</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Proper storage provided for bolts, nuts, washers, and other fastener components (Reference RCSC Specification, Section 2.2)</td>
<td>O</td>
<td>O</td>
</tr>
</tbody>
</table>

### TABLE NN5.6-2
**Inspection Tasks During Bolting**

<table>
<thead>
<tr>
<th>Inspection Tasks During Bolting</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastener assemblies, of suitable condition, placed in all holes and washers (if required) are positioned as required</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Joint brought to the snug-tight condition prior to the pretensioning operation</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Fastener component not turned by the wrench prevented from rotating</td>
<td>P</td>
<td>O</td>
</tr>
<tr>
<td>Fasteners are pretensioned in accordance with a method approved by the RCSC Specification and progressing systematically from the most rigid point toward the free edges</td>
<td>P</td>
<td>O</td>
</tr>
</tbody>
</table>

### Table NN5.6-3
**Inspection Tasks after Bolting**

<table>
<thead>
<tr>
<th>Inspection Tasks after Bolting</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Document acceptance or rejection of bolted connections</td>
<td>P</td>
<td>O</td>
</tr>
</tbody>
</table>
7. Other Inspection Tasks

The fabricator’s QAI shall inspect the fabricated steel to verify compliance with the details shown on the shop drawings, such as proper application of joint details at each connection. The erector’s QAI shall inspect the erected steel frame to verify compliance with the details shown on the erection drawings, such as braces, stiffeners, member locations, and proper application of joint details at each connection.

The QAI shall be on the premises for inspection during the placement of anchor rods and other embedments supporting structural steel for compliance with the construction documents. As a minimum, the diameter, grade, type and length of the anchor rod or embedded item, and the extent or depth of embedment into the concrete, shall be verified prior to placement of concrete.

The QAI shall inspect the fabricated steel or erected steel frame, as appropriate, to verify compliance with the details shown on the construction documents, such as braces, stiffeners, member locations and proper application of joint details at each connection.

NN6. MINIMUM REQUIREMENTS FOR INSPECTION OF COMPOSITE CONSTRUCTION

Inspection of structural steel and steel deck used in composite construction shall comply with the requirements of this Chapter.

For welding of steel headed stud anchors, the provisions of AWS D1.1/D1.1M apply.

For welding of steel deck, observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the construction documents. All applicable provisions of AWS D1.3/D1.3M Structural Welding Code—Sheet Steel shall apply. Deck welding inspection shall include verification of the welding consumables, welding procedure specifications, welding procedure qualification for nonprequalified joints, qualifications of welding personnel prior to the start of the work, observations of the work in progress, and a visual inspection of all completed welds. For steel deck attached by fastening systems other than welding, inspection shall include verification of the fasteners to be used prior to the start of the work, observations of the work in progress to confirm installation in conformance with the manufacturer’s recommendations, and a visual inspection of the completed installation.

In Table NN6.1, the inspection tasks are as follows:

P—Perform these tasks for each steel element.
TABLE NN6.1
Inspection of Steel Elements of Composite Construction Prior to Concrete Placement

<table>
<thead>
<tr>
<th>Inspection of Steel Elements of Composite Construction Prior to Concrete Placement</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Placement and installation of steel deck</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Placement and installation of steel headed stud anchors</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>Document acceptance or rejection of steel elements</td>
<td>P</td>
<td>P</td>
</tr>
</tbody>
</table>

NN7. NONCONFORMING MATERIAL AND WORKMANSHIP

Identification and rejection of material or workmanship that is not in conformance with the construction documents is permitted at any time during the progress of the work. However, this provision shall not relieve the owner or the inspector of the obligation for timely, in-sequence inspections. Nonconforming material and workmanship shall be brought to the immediate attention of the fabricator or erector, as applicable.

Nonconforming material or workmanship shall be brought into conformance, or made suitable for its intended purpose as determined by the engineer of record.

Nonconformance reports shall remain open until a suitable resolution to the cause of the nonconformance has been identified and corrective action documented.

**User Note:** Nonconforming items should be segregated and controlled to prevent inadvertent use or installation.
Modify Appendix 1 of the Specification as follows.

Replace preamble with the following:

This appendix addresses design by inelastic analysis, in which consideration of the redistribution of member and connection forces and moments as a result of localized yielding is permitted.

N1.1. GENERAL PROVISIONS

Add the following as the last paragraph:

When inelastic analysis is used for design, attention shall be paid to the induced deflections of the structural steel member(s), as well as to the effects of such deflections on supported components such as piping, HVAC ducts and cable trays, to ensure that the components will be able to perform their intended functions.

User Note: Increased deflections resulting from the utilization of inelastic design may cause additional component loading, and may reduce component clearances (gaps) required to prevent vibration interaction.
APPENDIX N2
DESIGN FOR PONDING

_No changes to Appendix 2 of the Specification._
APPENDIX N3
DESIGN FOR FATIGUE

No changes to Appendix 3 of the Specification.
Modify Appendix 4 of the Specification as follows.

N4.1. GENERAL PROVISIONS

Add the following paragraphs:

The intended functions of the structure under design basis fire shall be stated in the licensing document. The provisions of Appendix N4 are for life safety associated with evacuation of building occupants in the event of a design-basis fire. This specification does not address either “Important to Safety” structural steel members or loading conditions associated with a facility fire.

Structural steel shall be fire protected to achieve the fire resistance rating as established by fire hazard analysis. Where engineering analysis is used for structural design for fire conditions, design material parameters at elevated temperatures during the design-basis fire event shall be those defined in Table NA-4.2.1 and Table NA-4.2.2. Other material parameter values may be used provided they are substantiated or verified by test. The possible increased deflection that may occur due to elevated temperatures shall be considered in the design.

N4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

N4.2.3.1. Thermal Elongation

Replace section with the following:

The coefficients of expansion shall be taken as follows:

(a) For structural and reinforcing steels: For calculations at temperatures above 150°F (65 °C), the coefficient of thermal expansion shall be 7.8 × 10^{-6}/°F (1.4 × 10^{-5}/°C).

(b) For normal weight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 5.5 × 10^{-6}/°F (9.9 × 10^{-6}/°C).

User Note: Table NA-4.2.1 is intended for carbon steel applications. For stainless steel and other alloy steels the user needs to establish appropriate values based upon testing or qualified references.

Modify Table NA-4.2.1 as follows:
<table>
<thead>
<tr>
<th>Steel temperature °F (°C)</th>
<th>$k_E = E(T)/E = G(T)/G$</th>
<th>$k_p = F_p(T)/F_y$</th>
<th>$k_y = F_y(T)/F_y$</th>
<th>$k_u = F_u(T)/F_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>68 (20)</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>*</td>
</tr>
<tr>
<td>200 (93)</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>*</td>
</tr>
<tr>
<td>400 (204)</td>
<td>0.90</td>
<td>0.80</td>
<td>1.00</td>
<td>*</td>
</tr>
<tr>
<td>600 (316)</td>
<td>0.78</td>
<td>0.58</td>
<td>1.00</td>
<td>*</td>
</tr>
<tr>
<td>750 (399)</td>
<td>0.70</td>
<td>0.42</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>800 (427)</td>
<td>0.67</td>
<td>0.40</td>
<td>0.94</td>
<td>0.94</td>
</tr>
<tr>
<td>1000 (538)</td>
<td>0.49</td>
<td>0.29</td>
<td>0.66</td>
<td>0.66</td>
</tr>
<tr>
<td>1200 (649)</td>
<td>0.22</td>
<td>0.13</td>
<td>0.35</td>
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</tr>
<tr>
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<tr>
<td>1600 (871)</td>
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<tr>
<td>2000 (1093)</td>
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<td>0.02</td>
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<td>2200 (1204)</td>
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<td>0.00</td>
</tr>
</tbody>
</table>

* Use ambient temperature properties.

Modify Table A-4.2.2 (delete reference to lightweight concrete and add footnote *):
### TABLE NA-4.2.2*
Properties of Concrete at Elevated Temperatures

<table>
<thead>
<tr>
<th>Concrete temperature °F (°C)</th>
<th>$k_c = f'_c(T)/f'_c$</th>
<th>$E_c(T)/E_c$</th>
<th>$\varepsilon_{cu}(T)$, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>68 (20)</td>
<td>1.00</td>
<td>1.00</td>
<td>0.25</td>
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<tr>
<td>200 (93)</td>
<td>0.95</td>
<td>0.93</td>
<td>0.34</td>
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<tr>
<td>400 (204)</td>
<td>0.90</td>
<td>0.75</td>
<td>0.46</td>
</tr>
<tr>
<td>550 (288)</td>
<td>0.86</td>
<td>0.61</td>
<td>0.58</td>
</tr>
<tr>
<td>600 (316)</td>
<td>0.83</td>
<td>0.57</td>
<td>0.62</td>
</tr>
<tr>
<td>800 (427)</td>
<td>0.71</td>
<td>0.38</td>
<td>0.80</td>
</tr>
<tr>
<td>1000 (538)</td>
<td>0.54</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

* At 1,000 °F, concrete starts to deteriorate rapidly and the strength of reinforcing steel will be affected. This shall be taken into account in the design.
APPENDIX N5
EVALUATION OF EXISTING STRUCTURES

Replace Appendix 5 of the Specification with the following:

This appendix applies to the evaluation of the strength and stiffness under static vertical (gravity) loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the *engineer of record* or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section NA3.1. This appendix does not address load testing for the effects of seismic and other dynamic loads.

The appendix is organized as follows:

N5.1. General Provisions
N5.2. Material Properties
N5.3. Evaluation by Structural Analysis
N5.4. Evaluation by Load Tests
N5.5. Evaluation Report

N5.1. GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings, or (b) determination of the design strength of a force resisting member or system. The evaluation shall be performed by structural analysis (Appendix N5.3), by load tests (Appendix N5.4), or by a combination of structural analysis and load tests, as specified in the contract documents. Where load tests are used, the *engineer of record* shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent deformation that could affect the integrity of the equipment and components supported by it or located in its vicinity during testing.

N5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

   The *engineer of record* shall determine the specific tests that are required from Appendix N5.2.2 through N5.2.6 and specify the locations where they are required. Where available, the use of applicable design documents is permitted to reduce or eliminate the need for testing.

2. Tensile Properties

   Tensile properties of members shall be considered in evaluation by structural analysis (Appendix N5.3) or load tests (Appendix N5.4). Such properties shall include the yield stress, tensile strength and percent elongation. Where available, certified
material test reports (CMTR) or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, is permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure. In nuclear facilities, the use of the actual properties from CMTR, certified report, and the results of tensile tests is permissible when it can be shown that (1) the coupons taken for CMTR or a certified report represent the structure being evaluated, and (2) the value selected is derived from a statistical analysis indicating high confidence level. If necessary, additional coupons from the as-built structure shall be tested to supplement the CMTR or certified report results, as directed by the engineer of record.

3. **Chemical Composition**

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from CMTR or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures is permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

4. **Base Metal Notch Toughness**

Where welded tension splices in heavy shapes and plates as defined in Section NA3.1d are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the provisions of Section NA3.1d. If the notch toughness so determined does not meet the provisions of Section NA3.1d, the engineer of record shall determine if remedial actions are required.

5. **Weld Metal**

When specified by the engineer of record, representative samples of weld metal shall be obtained. The engineer of record shall specify the nature of the tests to be performed.

6. **Bolts**

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 is permitted.

**N5.3. EVALUATION BY STRUCTURAL ANALYSIS**

1. **Dimensional Data**

All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross-section dimensions, thicknesses, and connection
details, shall be determined from a field survey. Alternatively, when available, it is permitted to determine such dimensions from applicable design documents with field verification of critical values.

2. **Strength Evaluation**

   Forces (load effects) in members and connections shall be determined by structural analysis applicable to the type of structure evaluated. The load effects shall be determined for the loads and factored load combinations stipulated in Section NB2, except those involving seismic or dynamic loads.

   The available strength of members and connections shall be determined from applicable provisions of Chapters NB through NK of the Nuclear Specification.

3. **Serviceability Evaluation**

   Where required, the deformations at service loads shall be calculated and reported.

**N5.4. EVALUATION BY LOAD TESTS**

1. **Determination of Live Load Rating by Testing**

   To determine the live load rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the engineer of record’s plan. In addition to the load-deformation monitoring, the structure shall be monitored and shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

   The tested design strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested design strength equal to $1.2D + 1.6L$, where $D$ is the nominal dead load and $L$ is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures, $L_r$, $S$ or $R$ as defined in ASCE/SEI 7, shall be substituted for $L$. More severe load combinations shall be used where required by applicable regulatory and enforcement authorities.

   Periodic unloading shall be considered once the service load level is attained and before the load combination $1.2D + 1.6L$ is placed on the structure. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining the maximum test load for one hour, that the deformation of the structure remains essentially unchanged. It is permissible to repeat the sequence if necessary to demonstrate compliance.

   Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test
the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

2. **Serviceability Evaluation**

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformations recorded.

**N5.5. EVALUATION REPORT**

After the evaluation of an existing structure has been completed, the *engineer of record* shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by structural analysis, by load testing, or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, material test reports, and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the required strength of the structure, including members and connections, is adequate to withstand the load combinations of either Section NB2.5 or NB2.6, whichever is applicable.
APPENDIX N6

STABILITY BRACING FOR COLUMNS AND BEAMS

No changes to Appendix 6 of the Specification.
APPENDIX N7

ALTERNATIVE METHODS OF DESIGN FOR STABILITY

No changes to Appendix 7 of the Specification.
APPENDIX N8

APPROXIMATE SECOND-ORDER ANALYSIS

No changes to Appendix 8 of the Specification.
COMMENTARY
on the Specification for
Safety-Related Steel Structures
for Nuclear Facilities

January 31, 2012

INTRODUCTION

The Specification for Safety-Related Steel Structures for Nuclear Facilities is intended to be complete for normal design usage in the design, fabrication, and erection of safety-related steel structures for nuclear facilities in conjunction with the AISC Specification for Structural Steel Buildings and Commentary (ANSI/AISC 360-10).

This Commentary is nonmandatory and furnishes background information and references for the benefit of the engineer seeking further understanding of the derivation and limits of the Nuclear Specification.

The Nuclear Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.
Modify Chapter A of the Specification Commentary as follows:

NA1. SCOPE

Replace section with the following:

The scope of the Nuclear Specification is broader than that of the AISC Specification that it replaces: N690-06, Specification for Safety-Related Steel Structures for Nuclear Facilities (AISC, 2006).

The Specification for Safety-Related Steel Structures in Nuclear Facilities, hereafter referred to as the Nuclear Specification, follows the lead of the 2010 AISC Specification for Structural Steel Buildings (AISC, 2010a), hereafter referred to as the Specification, and modifies the provisions of previous AISC Nuclear Specifications to make it compatible with the Specification.

The basic purpose of the provisions in the Nuclear Specification is the determination of the required and nominal strength of the members, connections and other components of steel building structures. The nominal strength is usually defined in terms of resistance to a load effect, such as axial force, bending moment, shear or torque, but in some instances it is expressed in terms of a stress. The Nuclear Specification provides two methods of design.

(1) Load and Resistance Factor Design (LRFD): The nominal strength is multiplied by a resistance factor, $\phi$, and the resulting design strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate LRFD load combinations.

(2) Allowable Strength Design (ASD): The nominal strength is divided by a safety factor, $\Omega$, and the resulting allowable strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate ASD load combination.

The Nuclear Specification uses the provisions of the Specification for determining the values of the nominal strengths according to the applicable limit states and lists the corresponding values of the resistance factor, $\phi$, and the safety factor, $\Omega$. The ASD safety factors are calibrated to give approximately the same structural reliability and the same component size as the LRFD method.

The Nuclear Specification is applicable to all structural steel members in nuclear facilities. Specifically excluded from the Nuclear Specification are the pressure retaining components, for example, pressure vessels, valves, pumps and piping. For the materials, design, fabrication and examination of plate and shell component
supports, readers are directed to the requirements of Subsection NF of Section III of the ASME Boiler and Pressure Vessel Code (ASME, 2011).

The 2010 AISC Seismic Provisions for Structural Steel Buildings (AISC, 2010b), hereafter referred to as the Seismic Provisions, is intended for the design and construction of steel members and connections in the seismic force resisting systems in buildings for which the required strengths resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response.

The required strengths of seismic force resisting systems in safety-related structures for nuclear facilities are determined from elastic analyses where energy dissipation in the inelastic range is neglected. Thus, in general, the Seismic Provisions are not applicable to the design of safety-related structures for nuclear facilities. However, the detailing requirements of Section A3 and Chapter D of the Seismic Provisions should be appropriately considered when designing for plastic analysis.

For the purposes of the Nuclear Specification, hollow structural sections (HSS) are assumed to have constant wall thickness and a round, square or rectangular cross section that is constant along the length of the member. HSS are manufactured by forming strip or plate to the desired shape and joining the edges with a continuously welded seam. Published information is available describing the details of the various methods used to manufacture HSS (Graham, 1965; STI, 1996).

The 2010 AISC Code of Standard Practice for Steel Buildings and Bridges (AISC, 2010c), hereafter referred to as the Code of Standard Practice, defines the practices that are the commonly accepted standards of custom and usage for structural steel fabrication and erection. As such, the Code of Standard Practice is primarily intended to serve as a contractual document to be incorporated into the contract between the buyer and seller of fabricated structural steel. Some parts of the Code of Standard Practice, however, form the basis for some of the provisions in the Nuclear Specification. Therefore, the Code of Standard Practice is referenced in selected locations in the Nuclear Specification to maintain the ties between those documents, where appropriate.

**NA3. MATERIAL**

*Modify this section as follows:*

1. **Structural Steel Materials**

*Add the following:*

The Charpy V-notch energy values in Table NA3.1 have been carried forward from the original version of this Nuclear Specification, N690-1984 (AISC, 1984), and are values that assure a level of toughness suitable for most applications subjected to suddenly applied impact loads. For certain extreme applications and for applications where the structure is designed to absorb significant energy through deformation, the designer should review these criteria for appropriateness.
1a. ASTM Designations

Add the following:

Plates. ASTM A167 (Types 301, 302 and 302B) (ASTM, 2009) material has a carbon content of 0.15 and relatively low chromium and nickel content, which creates a problem with hot cracking. Further, these materials are susceptible to severe sensitization, and therefore, will require a final annealing to redissolve the carbides.

Bars. The unmodified martensitic grade of ASTM A276 (ASTM, 2008) is not readily weldable. Martensitic steels are susceptible to excessive hardening with consequent risk of cracking during welding.

1c. Rolled Heavy Shapes

and

1d. Built-up Heavy Shapes

Add the following:

Heavy structural sections and plates with restrained weld joints that induce stresses in the through-thickness direction are susceptible to lamellar tearing. The factors that affect susceptibility to lamellar tearing include joint configuration, service stresses, material thickness, material properties, fabrication techniques, and fabrication local strains. Proper design, materials selection and specification, and fabrication techniques can prevent lamellar tearing.

Joint configuration is most important in prevention of lamellar tearing. Fabrication strains are the principal cause of lamellar tearing, although in some cases the tearing might not occur until initiated by service stresses. By avoiding highly restrained configurations, lamellar tearing can be minimized. If highly restrained configurations cannot be avoided, then specifying materials resistant to lamellar tearing and/or fabrication techniques that reduce fabrication strains should be considered.

The through-thickness tension testing acceptance criteria have been carried forward from the original N690-I984 (AISC, 1984) version. They establish acceptance criteria based on the properties in the rolling direction rather than an absolute value, thereby adjusting the acceptance criteria to the material properties, since the material properties can vary significantly over the range of materials permitted.

Some guidelines for minimizing potential problems are provided in Thornton (1973). The figures from that commentary illustrate the advantages of improved joint configuration. Additional information can also be found in Jones and Milek (1975) and Thornton (1973).

5. Consumables for Welding

Add the following:

Because nuclear facilities sometimes utilize stainless steel structural materials, AWS D1.6/D1.6M (AWS, 2007), AWS A5.4/A5.4M (AWS, 2006a) and AWS A5.9/A5.9M
(AWS, 2006b) have been added to the Nuclear Specification. Previous AISC Nuclear Specifications referenced ASME *Boiler and Pressure Vessel Code* Section IX for stainless welding, but with the availability of AWS D1.6/D1.6M, the reference to Section IX has not been incorporated in this version of the Nuclear Specification.

**NA4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS**

*Add the following:*

The use of *Code of Standard Practice*, Section 3.1 is acceptable. However, because of the stringent requirements for quality control and inspection in nuclear facilities, the additional requirements for construction specifications are necessary.

*Add the following section:*

**NA5. QUALITY ASSURANCE**

The “Quality Assurance” Section has been added to comply with the requirements of the authority having jurisdiction (AHJ). For design of safety-related structures, this provision has been clarified to require the designer to follow the latest code, ASME NQA-1 (ASME, 2008) and the NQA-1a Addenda (ASME, 2009), or other approved standards; these other approved standards would include ANSI N45.2 (ANSI, 1977) documents, which pertain to older nuclear plants.
CHAPTER NB
DESIGN REQUIREMENTS

Modify Chapter B of the Specification Commentary as follows:

NB2. LOADS AND LOAD COMBINATIONS

Replace section with the following:

1. Normal Loads

Dead and live loads form a generic category of normal loads. During initial design, the values of most of the piping loads and suspended system loads (HVAC, cable trays, etc.) are not available, and the load allowance for these items is included in L as an area-averaged load. Once the final attachment loads are determined, the initial load assumptions should be confirmed. When designing for weights or pressures from fluids, either existing in the building or due to hydrostatic heads, both cases (with fluid present or absent) should be evaluated in order to establish the governing load condition. When a detailed dynamic analysis is performed for crane systems, elevators, or other moving machinery, the resulting load with dynamic amplification may be used in lieu of the load increases (dynamic impact factors) specified in ASCE/SEI 7-10 (ASCE, 2010), or similar documents.

Sections NB2.1 and NB2.2 state that the snow load, S, and wind load, W, are as stipulated in ASCE/SEI 7-10 for Risk Category IV facilities. Risk Category IV facilities are defined in Table 1.5-1 of ASCE/SEI 7-10 as those for which continued function following the occurrence of a natural phenomena hazard is essential for public health and safety. For such facilities, ASCE/SEI 7-10 requires that the nominal load otherwise determined for ordinary buildings and other structures be increased by an importance factor. This importance factor is 1.2 for snow load. These increases are tantamount to requiring Risk Category IV facilities to be designed for 100-year mean recurrence interval snow events. The importance factor for wind loads has been deleted (from previous editions of ASCE/SEI 7) due to changes in new wind hazard maps.

4. Abnormal Loads

A design-basis accident may be postulated to result from:

(a) A break in any of the high-energy piping existing in the plant. This can create compartment pressurization, short-term high temperatures, and dynamic loads of reaction and/or impingement associated with the postulated pipe rupture.

(b) A break in a small line containing high-temperature fluids or steam. This would result in a long-term high temperature and associated pressure loading.
5. **Load and Resistance Factor Design (LRFD)**

The Nuclear Specification permits design for strength by either the load and resistance factor design (LRFD) method or the allowable strength design (ASD) method. The load combinations stem from a probability-based study of load combinations for design of nuclear power plants (Hwang et al., 1987). The probabilistic methodology in that study is consistent with that used to develop the probability-based load combination requirements appearing in ASCE/SEI 7-10 (ASCE, 2010), Galambos et al. (1982), and Ellingwood et al. (1982). The load statistics for operating and abnormal plant conditions were obtained from a consensus estimation survey of operating load in nuclear facilities (Hwang et al., 1983).

Load Combination NB2-4 for severe environmental loads includes the wind load, $W$, from Section 26 of ASCE/SEI 7-10 (ASCE, 2010). This wind load addresses extreme nontornadic wind effects from extratropical storms and hurricanes. Tornadic wind effects are defined by $W_t$, and are addressed in Load Combination NB2-7 for extreme environmental effects. The extreme environmental loads, $W_t$ and $E_s$, as specified in NUREG-0800 (NRC, 2007) and in 10 CFR Part 50 (Office of the Federal Register, 2010), are design-basis events and thus appear in the load combinations with load factors of unity.

Dynamic load effects should be considered with maximum values assumed acting simultaneously, unless actual time history analysis shows a different time-phase relationship, in which case loads may be combined as a function of time. Loads due to postulated accidents and natural phenomena often yield dynamic response of short duration and rapidly varying amplitude in the exposed structures and components. For some loading phenomena, accident analysis provides a definitive time history response and allows a straightforward addition of responses where more than one load is acting concurrently. In other cases, no specified time-phase relationship exists, either because the loads are random in nature or because the loads have simply been postulated to occur together (for example, loss of coolant accident and safe shutdown earthquake) without a known or defined coupling. Where a defined time-phase relationship is lacking, system designers have utilized several approaches to account for the potential interaction of the loads. One approach, the so-called absolute or linear summation (ABS) method, linearly adds the absolute values of the peak structural response due to the individual dynamic loads. A second approach, referred to as the square root of the sum of the squares (SRSS) method, yields a combined response equal to the square root of the sum of the squares of the peak responses due to the individual dynamic loads. Research conducted over the past two decades shows that this method of combining dynamic responses is conservative unless the structural responses are stochastically dependent. The SRSS method of load combination is acceptable to the U.S. Nuclear Regulatory Commission (NRC, 1980), contingent upon the performance of a linear elastic dynamic analysis. Thus, the loads from a
loss of coolant accident (LOCA) and a seismic event combined in Load Combination NB2-4 may be combined by the SRSS method, provided that the responses are determined by elastic analysis. However, this does not prohibit the use of more conservative load combination schemes. In all cases, resultant dynamic loads shall be combined absolutely, considering both maximum positive and negative values, with applicable static loads.

6. **Allowable Strength Design (ASD)**

The starting point for the development of load combinations for allowable strength design was the load combinations that appear in ANSI/AISC N690-2006. These load combinations and accompanying stress limit coefficients were re-examined in the light of recent advances in the Specification as well as the principal action-companion action load combination format followed in ASCE/SEI 7-10 (ASCE, 2010) and in Section NB2.5 of the Nuclear Specification. The allowable strength design load combinations and other considerations in Section NB2.6 stem from this re-examination.

**NB3. DESIGN BASIS**

1. **Required Strength**

   *Add the following paragraph:*

   When using plastic design, adequate attention should be paid to the induced deflections of the structural steel member(s) as well as the effect of such deflections on supported components, such as piping, HVAC ducts and cable trays. Increased deflections resulting from the utilization of plastic design may cause additional component loading and reduce component clearances (gaps) required to prevent vibration interaction.

3. **Design for Strength using Load and Resistance Factor Design (LRFD)**

   *Add the following paragraph:*

   The strength of steel decreases at elevated temperatures. Where the structural component or system is exposed to sustained temperature in excess of 250 °F (121 °C), the decrease should be taken into account in determining the design strength. Design values for steel strength at elevated temperature may be obtained from ASME Code Section II-Part D (ASME, 2011).

4. **Design for Strength using Allowable Strength Design (ASD)**

   *Add the following paragraph:*

   The strength of steel decreases at elevated temperatures. Where the structural component or system is exposed to sustained temperatures in excess of 250 °F (121 °C), the decrease should be taken into account in determining the allowable strength. Design values for steel strength at elevated temperatures may be obtained from ASME Code Section II-Part D (ASME, 2011).
9. Design for Serviceability

*Add the following:*

The elastic modulus of steel decreases at elevated temperatures. Where the structural component or system is exposed to sustained temperatures in excess of 250 °F (121 °C), the effect of this decrease on structural stiffness and deformations should be taken into account.

*Add the following section:*

15. Design Based on Ductility and Local Effects

This section has no counterpart in the Specification, but is necessary for structures governed by the provisions of the Nuclear Specification.

Section NB3.15 permits the load effects from impact or impulsive forces to be determined by inelastic analysis, provided that the limits in Table NB3.1 are imposed on the total strains or deformations and that the width-to-thickness ratios for elements in flexure or compression conform to the limits in Table NB3.2. The limits in Table NB3.1 are based on the following considerations:

1. **Axial Tension:** Steel members under axial tension exhibit a ductility equivalent to full strain at ultimate stress. In developing the permitted local ductility factor, the strain at ultimate stress has been assumed to equal one-half the minimum specified percentage elongation at fracture, a factor of safety of two has been applied to that limit, and the maximum permitted strain has been limited to 0.10.

2. **Flexure:** The ductility factor of 20 for closed sections is based on tests reported in Howland and Newmark (1953). For open sections, the ductility factor is reduced to 10 when flexure governs and 5 when shear governs. In order to achieve these ductility factors, local buckling and lateral buckling must be prevented by limiting width-to-thickness ratios and unbraced lengths of compression members.

3. **Axial Compression:** The strength of short \(F_y/F_c < 0.0225\) rolled or welded built-up columns is controlled by yielding rather than by buckling, and the maximum permitted ductility factor is 10. Also, in no case should the ductility limit be allowed to exceed \(e_u/e_y\). As the slenderness increases, buckling controls. Research (Norris et al., 1959) has indicated that for \(F_y/F_c > 0.221\), the ductility factor should not be taken to be greater than unity. Between the upper bound \(\mu = 10\) when \(F_y/F_c = 0.0225\) and lower bound \(\mu = 1\) when \(F_y/F_c = 0.221\), the ductility factor is permitted to vary inversely with \(F_y/F_c\).

At the rates of strain that are characteristic of certain impactive or impulsive loads, structural steels exhibit elevated yield strengths, while the strain at the onset of strain hardening and the tensile strength increase slightly. The modulus of elasticity remains nearly constant. Section NB3.15 permits an upward adjustment in the yield stress used to compute nominal strength, \(R_n\), for strain rate effects. Such increases are permitted in other standards. ACI 349, Appendix C (ACI, 2006) recommends dynamic
increase factors (DIF) of 1.20 for Grade 40 reinforcement and 1.10 for Grade 60 reinforcement. Similar DIF are recommended in ASCE’s *Structural Analysis and Design of Nuclear Plant Facilities* (ASCE, 1986) and in the U.S. NRC Standard Review Plan 3.6.2 (NRC, 2007). Section NB3.15 permits a 10% increase over the specified yield strength, in the absence of supporting experimental data.

Table NB3.2 is based upon *Seismic Provisions*, Table D1.1. The limiting width-to-thickness ratio has been conservatively selected, treating structural members as highly ductile members.
CHAPTER NI

DESIGN OF COMPOSITE MEMBERS

Modify Chapter I of the Specification Commentary as follows.

Add the following:

The concrete structures in nuclear facilities are designed and constructed using ACI 349-06 (ACI, 2006). Hence, the applicable requirements of ACI 349-06, instead of ACI 318-08 (ACI, 2008) have been included.
CHAPTER NJ
DESIGN OF CONNECTIONS

Modify Chapter J of the Specification Commentary as follows:

NJ1. GENERAL PROVISIONS

10. Limitations on Bolted and Welded Connections

Add the following:

The potential for full reversal of design load and the likelihood of inelastic deformations of members and/or connected parts necessitate that pretensioned bolts be used in bolted joints in the seismic force resisting system. However, earthquake motions are such that slip cannot be prevented in all cases, even with slip-critical connections. Accordingly, these provisions call for bolted joints to be proportioned as pretensioned bearing joints but with faying surfaces prepared as for Class A or better slip-critical connections. That is, bolted connections can be proportioned with available strengths as for bearing connections as long as the faying surfaces are still prepared to provide a minimum slip coefficient of 0.33. The resulting nominal amount of slip resistance will minimize damage in moderate seismic events. Additionally, sharing of the available strength between welds and bolts on the same faying surface is not permitted.

Tension or shear rupture, bolt shear rupture, and block shear rupture are examples of limit states that generally result in nonductile failure of connections. As such, these limit states are undesirable as the controlling limit state for connections that are part of the seismic force resisting system and/or are subjected to dynamic loads. Accordingly, it is required that these connections be configured such that a ductile limit state in the member or connection, such as yielding or bearing deformation, controls the available strength. The design documents should identify the connections that are subjected to seismic or dynamic loads, and also should identify the type of load, i.e., axial force, shear, moment or torsion.

NJ2. WELDS

6. Filler Metal Requirements

Add the following:

Additional notch toughness requirements have been incorporated. The provisions have been based on the Seismic Provisions.
NJ3. BOLTS AND THREADED PARTS

10. Bearing Strength at Bolt Holes

*Add the following:*

Since deformations are always a design consideration for nuclear structures, the nominal bearing strength is limited to $2.4t_F u$. 

Modify Chapter L of the Specification Commentary as follows:

NL1. GENERAL PROVISIONS

Replace section with the following:

The General Provisions for serviceability for a nuclear plant structure differ from those in the Specification. For nuclear plant structures, the focus on serviceability is on the ability of safety-related structures to perform under their intended design conditions that are described in various licensing documents. Deflection and vibration are a primary concern for safety-related structures due to the ramifications that these deflections and vibrations may have on adjacent safety-related systems and components. Due to the robustness of nuclear plant structures, the comfort of the occupants is generally not an issue; accordingly the Specification Commentary referral to ASCE/SEI 7 (ASCE, 2010) is not applicable.
CHAPTER NM
FABRICATION AND ERECTION

Modify Chapter M of the Specification Commentary as follows:

NM2. FABRICATION

4. Welded Construction

Add the following:

Because nuclear facilities sometimes utilize stainless steel structural materials, AWS D1.6/D1.6M (AWS, 2007) has been added to the Nuclear Specification.

NM3. SHOP PAINTING

Add the following:

Because painting and associated quality and documentation requirements for nuclear facilities vary widely depending on the facility and location in the facility, it is not practical to cover them in the Nuclear Specification and coverage is left to the individual project specifications.

NM4. ERECTION

Add the following new section:

7. Tolerances for Cranes

The CMAA Specification tolerances have been adopted where appropriate. The criteria for column base lines, crane runway girders, and rail eccentricity provide tolerances not prescribed by the CMAA Specification (CMAA, 2007). These additional tolerances, which have evolved in the Nuclear Specification, minimize secondary effects onto the building structure and provide assurance of additional quality control required in a nuclear facility.
CHAPTER NN
QUALITY ASSURANCE AND QUALITY CONTROL

Replace Chapter N of the Specification Commentary with the following:

Because of the unique quality assurance requirements applicable to nuclear facilities, the fabricator’s quality assurance and control procedures must meet the regulatory requirements as invoked by the purchaser through their specifications.

Chapter NN of the Nuclear Specification is a stand-alone chapter that, while based upon the Specification, is unique due to the regulatory requirements for nuclear facilities.

ASME NQA-1 and NQA-1a Addenda (ASME, 2008, 2009) stipulates the requirements for the establishment and execution of quality assurance programs for nuclear facilities. Quality Assurance programs are pertinent to the designer, engineer, material supplier, fabricator, erector and constructor, and each entity is required to establish such a program. The provisions of the Nuclear Specification are intended to supplement the NQA-1 requirements.

Subpart 2.4 of ASME NQA-1 and NQA-1a Addenda (ASME, 2008, 2009) establishes installation, inspection and testing requirements for various structural items, including structural steel.

The Nuclear Specification’s usage of the terms quality assurance and quality control differ from the Specification. A quality assurance program includes the planned or systematic actions necessary to provide adequate confidence that an item or facility will be designed, fabricated, erected or constructed in accordance with the plans and specification. Quality control is a process employed by the fabricator, erector or constructor to verify that the item or facility is fabricated, erected or constructed in accordance with the plans and specification.
APPENDIX N4
STRUCTURAL DESIGN FOR FIRE CONDITIONS

Modify Appendix 4 of the Specification Commentary as follows:

N4.1. GENERAL PROVISIONS
Add the following:

Material properties at elevated temperatures included in the specification cover structural steel commonly used as defined in the Specification (AISC, 2010a). For other steels such as stainless steel and forging steel, suitable properties should be obtained based on reliable test results. It should be also pointed out that the material properties at elevated temperatures are short-term properties intended for fire design by analysis only. They should not be used in assessing the long-term performance of structural steel subjected to elevated temperature.
Replace Appendix 5 of the Specification Commentary with the following:

N5.1. GENERAL PROVISIONS

The load combinations referred to in this chapter pertain to gravity loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, such as lateral loads, the appropriate load combination from Section NB2 should be used. The engineer of record for a project is generally established by the owner.

N5.2. MATERIAL PROPERTIES

6. Bolts

Because connections typically are more reliable than structural members, removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they cannot be properly identified otherwise.
REFERENCES

The References listed below are in addition to those in the AISC Specification for Structural Steel Buildings.

ACI (2006), Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary, ACI 349-06, American Concrete Institute, Farmington Hills, MI.

ACI (2008), Building Code Requirements for Structural Concrete, ACI 318-08 and ACI 318M-08, American Concrete Institute, Farmington Hills, MI.


AISC (2010a), Specification for Structural Steel Buildings, ANSI/AISC 360-10, June 22, American Institute of Steel Construction, Chicago, IL.

AISC (2010b), Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-10, June 22, American Institute of Steel Construction, Chicago, IL.

AISC (2010c), Code of Standard Practice for Steel Buildings and Bridges, American Institute of Steel Construction, Chicago, IL.

ANSI (1977), Quality Assurance Program Requirements for Nuclear, ANSI N45.2, American National Standards Institute, Washington, DC.

ASCE (1986), Structural Analysis and Design of Nuclear Plant Facilities, American Society of Civil Engineers, Reston, VA.

ASCE (2010), Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10, American Society of Civil Engineers, Reston, VA.


AWS (2006a), Specification for Stainless Steel Electrodes for Shielded Metal Arc Welding, AWS A5.4/5.4M, American Welding Society, Miami, FL.
REFERENCES

AWS (2006b), *Specification for Bare Stainless Steel Welding Electrodes and Rods*, AWS A5.9/A5.9M, American Welding Society, Miami, FL.


STI (1996), *Principal Producers and Capabilities*, Steel Tube Institute, Mentor, OH.
