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Specification for

Safety-Related Steel Structures

for Nuclear Facilities

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION
130 East Randolph Street, Suite 2000, Chicago, Illinois 60601
www.aisc.org

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Only revisions that are shown with tracked changes in Chapter NA, Chapter NM, Appendix N9, and Appendix N10 are open for public comments at this time. The rest of the draft is provided for context.

You will find revisions on the following pages: (also bookmarked in the PDF):
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App N10 Sec.--Page 119

SYMBOLS

Definitions for the symbols used in this standard are provided here and reflect the definitions provided in the body of this standard. Some symbols may be used multiple times throughout the document. The section or table number shown in the right-hand column of the list identifies the first time the symbol is used in this document. Symbols without text definitions are omitted.

Symbol	Definition	Section
A_c	Area of concrete infill per unit width, in.^{2}/ft (mm^2/m)	App. N9.2.2
A_g	Gross area of member, in.^{2} (mm^2)	Table A-N10.2.2
A_s	Gross area of faceplates per unit width, in.^{2}/ft (mm^2/m)	App. N9.1.1
A_s^F	Gross area of faceplate in tension due to flexure per unit width, in.^{2}/ft (mm^2/m)	App. N9.3.3
A_{sn}	Net area of faceplates per unit width, in.^{2}/ft (mm^2/m)	App. N9.1.1
C	Rated capacity of crane	NB2.1
D	Dead loads due to weight of the structural elements, fixed-position equipment, and other permanent appurtenant items; weight of crane trolley and bridge	NB2.1
D	Outside diameter of round HSS, in. (mm)	Table A-N10.2.1
D_m	Maximum deflection from analysis, in. (mm)	App. N10.2.4
D_{tie}	Equivalent diameter of shear reinforcement, in. (mm)	App. N9.1.5b
D_y	Effective yield deflection, in. (mm)	App. N10.2.4
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	Table A-N10.2.1
E_c	Modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f_c'}$, ksi, ($0.043 w_c^{1.5}\sqrt{f_c'}$, MPa)	Table NA-4.2.2
$E_c(T)$	Modulus of elasticity of concrete at elevated temperature, ksi (MPa)	Table NA-4.2.2
E_m	As-modeled material elastic modulus used in elastic finite element analysis of SC panel section, ksi (MPa)	App. N9.2.3
E_o	Loads generated by operating basis earthquake	NB2.2
E_s	Loads generated by the safe shutdown earthquake, or design basis earthquake	NB2.3
E_s	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa) for carbon steel and duplex stainless steel = 28,000 ksi (193 000 MPa) for austenitic stainless steel	App. N9.1.3
$(EI)_{eff}$	Effective flexural stiffness for analysis of SC structural elements per unit width, kip-in.^{2}/ft (N-mm^2/m)	App. N9.2.2
$(EI)_{eff}$	Effective SC stiffness per unit width used for buckling evaluation, kip-in.^{2}/ft (N-mm^2/m)	App. N9.3.2
F	Loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights	NB2.1
F_e	Elastic buckling stress, ksi (MPa)	Table A-N10.2.2
F_{nr}	Nominal rupture strength of the tie, or the nominal strength of the	

associated welded or threaded connection, whichever is smaller, kips (N)....App. N9.1.5a

F_{ny} Nominal yield strength of the tie based on its gross area if no threads are present, and on its root area if it is threaded, kips (N)....................................App. N9.1.5a

F_t Nominal tensile strength of tie, kips (N)..……….........................App. N9.3.5

F_u Specified minimum tensile strength, ksi (MPa)…....................................NJ3.11

F_y Specified minimum yield stress, ksi (MPa). As used in this Specification, “yield stress” denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point) …..N9.1

$(GA)_{eff}$ Effective in-plane shear stiffness per unit width, kip/ft (N/m) ....App. N9.2.2

$(GA)_{uncr}$ In-plane shear stiffness per unit width of uncracked composite SC panel section, kip/ft (N/m)…………………………….......................App. N9.2.2

G_c Shear modulus of concrete, ksi (MPa)………..…………………..App. N9.2.2

G_s Shear modulus of elasticity of steel ....App. N9.2.2
= 11,200 ksi (77 200 MPa) for carbon steel and duplex stainless steel
= 10,800 ksi (74 500 MPa) for austenitic stainless steel

H Loads due to weight and pressure of soil, water in soil, or bulk materials.............. ....NB2.1

I_c Moment of inertia of concrete infill per unit width, in.^{4}/ft (mm^4/m)..App. N9.2.2

I_s Moment of inertia per unit width of faceplates (corresponding to the condition when the concrete is fully cracked), in.^{4}/ft (mm^4/m)…… .... App. N9.2.2

K_p Strength-dependent concrete penetrability factor………………….App. N10.3.2

K_{psc} Penetration depth modification factor for SC cross section……….App. N10.3.2

L Live load due to occupancy and moveable equipment, including impact...NB2.1

L_c Effective length of member, in. (mm)… ....App. N9.3.2

L_d Development length, in. (mm)…………………….……………….App. N9.1.4b

L_r Roof live load...NB2.1

M_n Nominal flexural strength per unit width, kip-in./ft (N-mm/m)..……App. N9.1.4b

M_{rx}, M_{ry} Required out-of-plane flexural strength per unit width, kip-in./ft (N-m/m)……….. ....App. N9.2.4

M_{rxy} Required twisting moment strength per unit width, kip-in./ft (N-mm/m) ....App. N9.2.5

N Missile nose shape factor per the modified NDRC formula……….App. N10.3.2

P_a Maximum differential pressure load generated by postulated accident NB2.4

P_{ci} Available compressive strength per unit width for each notional half of SC panel section, kip/ft (N/m)…..……...................App. N9.3.6b

P_e Elastic critical buckling load per unit width, kip/ft (N/m)..................App. N9.3.2

P_{no} Nominal compressive strength per unit width, kip/ft (N/m)………... App. N9.3.2

Q_{cv} Available shear strength of shear connector, kips (N) .... App. N9.1.4a

Q_{cv}^{avg} Weighted average of the available interfacial shear strengths of a group of shear connectors that accounts for tributary areas of each type of connector, kips (N)

.... App. N9.1.4b

R Rain load NB2.1

R_a Pipe and equipment reactions generated by a postulated accident, including R_o NB2.4

R_a Required strength using ASD load combinations …… NB2.6

R_n Nominal strength….. NB2.5

R_o Pipe reactions during normal operating, start-up or shutdown conditions, based on most critical transient or steady-state condition NB2.1

R_u Required strength using LRFD load combinations ……................................ NB2.5

R_y Ratio of the expected yield stress to the specified minimum yield stress, F_y, of that material……… .... App. N10.2.1

S Snow load……………………………………………………................... NB2.1

S_{cr} In-plane shear force per unit width at concrete cracking threshold, kip/ft (N/m) .... App. N9.2.2

$S_{r,max}$ Maximum required principal in-plane strength per unit width for notional half of SC panel section, kip/ft (N/m)................................ App. N9.3.6b

$S_{r,min}$ Minimum required principal in-plane strength per unit width for notional half of SC panel section, kip/ft (N/m)................................ App. N9.3.6b

S_{rx} Required membrane axial strength per unit width in direction x, kip/ft (N/m) .... App. N9.2.5

S_{rxy} Required membrane in-plane shear strength per unit width, kip/ft (N/m).………………………. App. N9.2.2

S_{ry} Required membrane axial strength per unit width in direction y, kip/ft (N/m) .... App. N9.2.5

S'_{rx} Required membrane axial strength per unit width in direction x for each notional half of SC panel section, kip/ft (N/m) .... App. N9.3.6b

S'_{rxy} Required membrane in-plane shear strength per unit width for each notional half of SC panel section, kip/ft (N/m)………….. App. N9.3.6b

S'_{ry} Required membrane axial strength per unit width in direction y for each notional half of SC panel section, kip/ft (N/m) .... App. N9.3.6b

T_a Thermal loads generated by a postulated accident, including T_o................. NB2.4

T_{ci} Available tensile strength per unit width for each notional half of SC panel section, kip/ft (N/m).. App. N9.3.6b

T_{ni} Nominal tensile strength per unit width of SC panel section, kip/ft (N/m).. App. N9.3.6b

T_o Thermal effects and loads during normal operating, start-up or shutdown conditions, based on most critical transient or steady-state condition......... NB2.1

V_c Available out-of-plane shear strengths per unit width of SC panel section in local x (V_{cx}) and y (V_{cy}) directions, kip/ft (N/m)............... App. N9.3.6a

$V_{c\ conc}$ Available out-of-plane shear strength contributed by concrete per unit width of SC panel section, kip/ft (N/m)................................... App. N9.3.6a

V_{ci} Available in-plane shear strength per unit width for each notional half of SC panel section, kip/ft (N/m).…………................……. App. N9.3.6b

V_{conc} Nominal out-of-plane shear strength contributed by concrete per unit width of SC panel section, kip/ft (N/m)…................................... App. N9.3.5

V_i Initial (pre-impact) velocity of missile, ft/s (m/s)………………….App. N10.3.2

V_{ni} Nominal in-plane shear strength per unit width of SC panel section, kip/ft (N/m)…..….. App. N9.3.4

V_{no} Nominal out-of-plane shear strength per unit width of SC panel section, kip/ft (N/m)…..….. App. N9.3.5

V_p Perforation velocity for reinforced concrete section of same thickness, ft/s (m/s)………………………………………………….App. N10.3.2

V_r Required out-of-plane shear strength per unit width of SC panel section in local x (V_{rx}) and y (V_{ry}) directions using LRFD or ASD load combinations, kip/ft (N/m) ....App. N9.3.6a

V_r Residual velocity of a missile passing through concrete, ft/s (m/s).App. N10.3.2

V_{rx} Required out-of-plane shear strength per unit width along edge parallel to direction x, kip/ft (N/m) ....App. N9.2.5

V_{ry} Required out-of-plane shear strength per unit width along edge parallel to direction y, kip/ft (N/m) ....App. N9.2.5

W Wind load..NB2.2

W_{cf} Weight of the concrete frustum (plug) associated with x_{sc}, the penetration depth, of the impacting missile, lb (N)………………………App. N10.3.2

W_p Missile weight, lb (N)…………………………………………….App. N10.3.2

W_t Loads generated by the specified design (basis) tornado, including wind pressures, pressure differentials, and tornado-borne missiles.....................NB2.3

Y_j Jet impingement load generated by the postulated accident.......................NB2.4

Y_m Missile impact load, such as pipe whip generated by or during the postulated accident NB2.4

Y_r Loads on structure generated by reaction of a broken high-energy pipe during a postulated accident NB2.4

b Largest unsupported length of the faceplate between rows of shear connectors, in. (mm)...NM2.7

b Width of compression element as shown in Table A-N10.2.1, in. (mm) .... Table A-N10.2.1

c_2 Calibration constant for determining effective flexural stiffness ....App. N9.2.2

c_m As-modeled specific heat used in elastic finite element analysis of SC panel section, Btu/lb-°F (J/kg-°C)…… ....App. N9.2.3

d Full depth of the section, in. (mm)…... Table A-N10.2.1

d Nominal diameter of fastener, in. (mm)…..NJ3.11

d Effective diameter of the missile, in (mm)…………………………App. N10.3.2

f_w Faceplate waviness...NM2.7

f'_c Specified compressive strength of concrete, ksi (MPa)….............. Table NA-4.2.2

$f'_c(T)$ Specified compressive strength of concrete at elevated temperature, ksi (MPa) ……... Table NA-4.2.2

g acceleration due to gravity………………………………………App. N10.3.2 = 386.4 in./s^2 (9.81 m/s^2)

h Width of compression element as shown in Table A-N10.2.1, in. (mm) .... Table A-N10.2.1

j_x, j_y Parameter for distributing required flexural strength into the corresponding membrane force couple acting on each notional half of SC panel section..App. N9.3.6b

j_{xy} Parameter for distributing required flexural strength, M_{rxy}, into the corresponding membrane force couples acting on each notional half of SC panel section.......................................App. N9.3.6b

k_m As-modeled thermal conductivity used in the elastic finite element analysis of SC panel sections, Btu/ft-sec-ºF (W/m-ºC)……………………..App. N9.2.3

l_c Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)……..…NJ3.11

n Modular ratio of steel and concrete……………………………..… ..App. N9.2.2

r Radius of gyration, in. (mm)…... Table A-N10.2.2

r_1 effective radius of the missile, in. (mm)…………………………… App. N10.3.2

r_2 concrete frustum radius at penetration depth equal to x_{sc}, in. (mm)…………………………………………………………. App. N10.3.2

s Spacing of steel anchors, in. (mm)……………….................................... NM2.7

s_{tl} Spacing of ties in the longitudinal direction, in. (mm) ....App. N9.1.5b

$s_{t,min}$ Minimum tie spacing, in. (mm) NM2.7

s_{tt} Spacing of ties in the transverse direction, in. (mm) ....App. N9.1.5b

t Thickness of connected material, in. (mm)…... NJ3.11

t Thickness of element as shown in Table A-N10.2.1, in. (mm) …... .... Table A-N10.2.1

t_c Concrete infill thickness, in. (mm)…………………….…..............App. N9.2.2

t_m As-modeled section thickness of SC panel section, in. (mm)……....App. N9.2.3

t_p Thickness of faceplate, in. (mm).. NM2.7

$t_{p,\ min}$ Minimum required thickness of faceplate, in. (mm)......................App. N10.3.2

t_{sc} SC section thickness, in. (mm)………….....……………… ....App. N9.1.1

t_w Thickness of web, in. (mm)….. .... Table A-N10.2.1

x_c Concrete penetration depth for the reinforced concrete section of the same thickness as the SC cross section, calculated using the modified NDRC formula, in. (mm)……………………………………………………….App. N10.3.2

x_{sc} Missile penetration depth into the SC cross section, in. (mm)……..App. N10.3.2

ΔT_{savg} Average of the maximum surface temperature increases for the faceplates due to accident thermal conditions, ºF (ºC)……. .... App. N9.2.2

ΔT_{sg} Maximum temperature difference between faceplates due to accident thermal conditions in ºF (ºC).......................................…. App. N9.2.4b

θ Inclination angle of the concrete frustum extending from the penetration depth of the impacting missile to the back faceplate of the impacted SC section… .....App. N10.3.2

Ω Safety factor…..NB2.6

Ω_{ci} Safety factor for compression for each notional half……………… App. N9.3.6b

Ω_{ti} Safety factor for tension for each notional half…………………... App. N9.3.6b

Ω_{vi} Safety factor for in-plane shear………………………………..… App. N9.3.4

Ω_{vo} Safety factor for out-of-plane shear………………………...……. App. N9.3.5

Ω_{vs} Safety factor for in-plane shear for each notional half……………. App. N9.3.6b

α Ratio of available in-plane shear strength to available tensile strength for each notional half of SC panel section….……… ....App. N9.3.6b

α_m As-modeled thermal expansion coefficient used in the elastic finite element analysis of SC panel section, °F^{-1} (°C^{-1})………………… ..App. N9.2.3

α_p	Missile deformability factor………………….................................App. N10.3.2
α_s	Thermal expansion coefficient of faceplate, °F^{-1} (°C^{-1})……….......App. N9.2.4b
β	Ratio of available in-plane shear strength to available compressive strength for each notional half of SC panel section………….. App. N9.3.6b
γ_m	As-modeled material density used in elastic finite element analysis of the SC panel section, lb/ft^3 (kg/m^3)……………………...App. N9.2.3
ξ	Factor used to calculate shear reinforcement contribution to out-of-plane shear strength (depends on whether the shear reinforcement is yielding or nonyielding type)…. ……....…App. N9.3.5
$\varepsilon_{cu}(T)$	Concrete strain corresponding to $f'_c(T)$ at elevated temperature, in./in. (mm/mm) ….. Table NA-4.2.2
ε_{st}	Strain corresponding to the onset of strain hardening .... Table A-N10.2.2
ε_u	Strain corresponding to elongation at failure (rupture).............. Table A-N10.2.2
ε_y	Strain corresponding to nominal yield stress .... Table A-N10.2.2
μ_p	Permissible ductility ratio ....App. N10.2.4
μ_r	Required ductility ratio ....App. N10.2.4
ν_m	As-modeled poisson’s ratio used in elastic finite element analysis of the SC panel section...App. N9.2.3
ρ	Reinforcement ratio…………………………………..……… .App. N9.1.1
ρ_c	Concrete density, lb/ft^3 (kg/m^3)……………………..………. .....App. N10.3.2
$\overline{\rho}$	Strength-adjusted reinforcement ratio……..………………………...App. N9.2.2
ρ'	Stiffness-adjusted reinforcement ratio…………….………………. ...App. N9.2.2
σ_r	Equivalent radial compressive stress in the rear faceplate, based on von Mises yield criterion, ksi (MPa)…………………………………..………App. N10.3.2
ϕ	Resistance factor…… NB2.5
ϕ_{ci}	Resistance factor for compression for each notional half………......App. N9.3.6b
ϕ_{ti}	Resistance factor for tension for each notional half….…………….…App. N9.3.6b
ϕ_{vi}	Resistance factor for in-plane shear………………………..…… App. N9.3.4
ϕ_{vo}	Resistance factor for out-of-plane shear ....App. N9.3.5
ϕ_{vs}	Resistance factor for in-plane shear for each notional half………. App. N9.3.6b

GLOSSARY

The terms listed below shall be used in addition to or replacements for those in the AISC *Specification for Structural Steel Buildings*.

Analysis calculation. Document detailing the process used to determine the required strength and anticipated settlements and deflections of a structure under the applied loads.

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.

Connection region. A designated strip along the edge of any two intersecting structural elements (for example, slabs, walls and basemats) where force transfer between the connected elements is required to be accomplished.

Dedication. The process in which critical characteristics for a commercially obtained material or component are identified and validated for use in safety-related applications by inspections, testing, or analyses.

Design basis earthquake (or) design/evaluation basis earthquake (DBE). See *safe shutdown earthquake* (*SSE*). Term used in connection with U.S. Department of Energy (DOE) facilities; also used interchangeably for older nuclear power facilities.

Design calculation. Document detailing the process used to proportion the members, connections, and structure to have adequate available strength, constructability, and serviceability.

Design documents. Analysis and design calculations, design drawings, design models, or a combination of drawings and models as well as construction specifications. In this Specification, reference to these design documents indicates design documents that are issued for construction.

Ductile limit state. Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width-to-thickness limitations of Table NB3.4. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.

Dynamic increase factor (DIF). Factor that accounts for increase in nominal yield strength of the material for loading applied at high strain rates (i.e., impulsive and impactive loads).

Dynamic load factor (DLF). Amplification factor applied to the peak (positive or negative) load to account for the dynamic effects of impulsive and impactive loads.

Effective flexural stiffness. Cracked transformed flexural stiffness of the steel-plate composite structural element used for elastic finite element analysis.

Effective in-plane shear stiffness. Cracked transformed shear stiffness of the steel-plate composite structural element used for elastic finite element analysis.

Effective steel-plate composite (SC) stiffness. Effective stiffness of the steel-plate composite panel section used for buckling evaluation.

Engineer of record (EOR). 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 is a licensed professional engineer, qualified to fulfill the assigned responsibility.

Faceplates. The two exterior steel plates of a steel-plate composite structural element (slab, wall, or basemat) that serve as its reinforcement.

Faceplate waviness. The waviness of steel-plate composite module faceplates after concrete curing, measured as the distance of the lowest point (trough) from the straight line joining two adjacent high points (crests).

Impactive force. Time-dependent loads due to the collision of solid masses that are associated with finite amounts of kinetic energy, where the impactive load is determined by the inertia and stiffness properties of the impactor and the target structure.

Impulsive force. Time-dependent load (force or pressure) for which the rate of loading and its duration affect the structural response.

Interior region. Region of steel-plate composite structural element that is bounded by the designated connection region strips.

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.

Large opening. Openings in steel-plate composite structural elements with the largest dimension greater than half the section thickness.

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

Module. A combination of sub-modules.

Nonyielding shear connector. Shear connector that does not meet the requirements of a yielding shear connector per Section N9.1.4a.

Nonyielding shear reinforcement. Ties that do not meet the requirements of yielding shear reinforcement.

No paint area. Defined area on a member within which painting or coating is prohibited until the field weld designated for that location has been completed.

Notional half. Each half of the steel-plate composite panel section consisting of one faceplate and half the concrete thickness.

Operating basis earthquake (OBE). Earthquake that produces vibratory ground motion for which those features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public will remain functional. Unless elected by the Owner as a design input, the OBE is only associated with plant shutdown and inspection.

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

Panel. Basic shippable modular unit; typically fabricated in the shop and then shipped to the field.

Panel section. The extent of the steel-plate composite structural element over which the demands are averaged to calculate the required strengths.

Permissible ductility ratio. Ratio of maximum permitted inelastic deflection to the deflection at the effective yield point on the idealized bilinear elastic-plastic force-deflection diagram.

Plastic instability. Member response that is characterized by a limit state of sustained negative stiffness in the stress-strain or load-deflection curve.

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

Quality assurance inspector (QAI). Individual(s) designated to independently 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 specifications.

Quality control inspector (QCI). Individual(s) designated to provide quality control inspection for the work being performed.

Required ductility ratio. The ratio of maximum inelastic strain (or deflection) to the effective yield strain (or deflection) obtained by performing inelastic analysis considering bilinear (or multilinear) stress-strain (or force-deflection) behavior.

Rib. Steel section used to increase faceplate stiffness and strength to handle rigging and construction loads (for example, wet concrete pressure) before the concrete hardens and to serve as a shear connector thereafter.

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 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 ensure:
(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.

Shear connector. Embedded structural steel element in steel-plate composite construction, such as a rib, steel headed stud anchor, anchor made of a shape or plate, and a tie, that enables composite action between concrete infill and steel faceplates.

Steel-plate composite (SC) structural element. A structural element consisting of two steel faceplates acting compositely with structural concrete infill, where the faceplates are connected together with ties and, if needed, additional shear connectors.

Section thickness. The total thickness of the steel-plate composite panel section.

Small opening. An opening in the steel-plate composite structural element with the largest dimension not greater than half the section thickness.

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, U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.76).

Sub-module. A combination of panels in a co-planar, L-shaped, T-shaped, corner, or any other pattern that is suitable for further assembly into a module.

Tie. Discrete structural component such as a steel shape, frame, or bar that connects two faceplates of an steel-plate composite element together at regular intervals.

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.

Yielding shear reinforcement. Ties with nominal yield strength less than or equal to 0.85 times the nominal rupture strength and 0.85 times the nominal strength of the associated connection.

ABBREVIATIONS

The following abbreviations appear in this *Nuclear Specification.* The abbreviations are written out when they first appear within a Section.

ABC (applicable building code)
ACI (American Concrete Institute)
AHJ (authority having jurisdiction)
AISC (American Institute of Steel Construction)
AISI (American Iron and Steel Institute)
ANSI (American National Standards Institute)
ASCE (American Society of Civil Engineers)
ASD (allowable strength design)
ASME (American Society of Mechanical Engineers)
ASNT (American Society for Nondestructive Testing)
ASTM (ASTM International)
AWI (associate welding inspector)
AWS (American Welding Society)
CFR (U.S. Code of Federal Regulations)
CJP (complete joint penetration)
CMAA (Crane Manufacturers Association of America)
CMTR (certified material test report)
CVN (Charpy V-notch)
DBE (design basis earthquake or *design/evaluation basis earthquake)*
DIF (dynamic increase factor)
DLF (dynamic load factor)
DOE (U.S. Department of Energy)
EOR (engineer of record)
EPRI (Electric Power Research Institute)
HSS (hollow structural section)
HVAC (heating, ventilation and air conditioning)
LOCA (loss-of-coolant accident)
LRFD (load and resistance factor design)
MT (magnetic particle testing)
NDE (nondestructive examination)
NRC (U.S. Nuclear Regulatory Commission)
OBE (operating basis earthquake)
PJP (partial joint penetration)
PQR (procedure qualification record)
PT (penetration testing)
QA (quality assurance)
QAI (quality assurance inspector)
QC (quality control)
QCI (quality control inspector)
RC (reinforced concrete)
RCSC (Research Council on Structural Connections)

RT (radiographic testing)
SC (steel-plate composite)
SEI (Structural Engineering Institute)
SSE (safe shutdown earthquake)
SWI (senior welding inspector)
UT (ultrasonic testing)
WI (welding inspector)
WPQR (welding personnel performance qualification records)
WPS (welding procedure specification)

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 for 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 Documents and Specifications
NA5. Approvals
NA6. 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, fabrication, erection, and quality 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 Symbols, Glossary terms, Abbreviations, Chapters NA through NN, and Appendices N1 through N10. The Commentary and User Notes interspersed throughout the Nuclear Specification are not part of the Nuclear Specification. The phrases "is permitted" and "are permitted" in this document identify provisions that comply with the Nuclear Specification, but are not mandatory.

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

> **User Note:** With the exception of Appendix N9, this standard does not include seismic detailing requirements for safety-related nuclear structures constructed using structural steel and composite members. The authority having jurisdiction may adopt the pertinent requirements in ASCE 43.
>
> For SC structural elements and their connections, the design and detailing requirements specified in Appendix N9 are adequate for seismic applications.

The steel elements shall be as defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303), 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 and structural elements that are part of a safety-related system or that support, house or protect safety-related systems or components, the failure of which could credibly result in the loss of capability of the structure, system or component to perform its safety functions. Concrete that is part of composite members and steel-plate composite (SC) structural elements is also subject to the Nuclear Specification. 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, including, but not limited to, pressure vessels, valves, pumps, and piping.

When designing for inelastic behavior such as that caused by impact loads, the design shall follow the material requirements of Section A3 of the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341), hereafter referred to as the *Seismic Provisions,* and the general member and connection requirements of *Seismic Provisions* Sections D1 and D2 for highly ductile members, respectively.

For a structural system or construction within the scope of the Nuclear Specification where conditions are not covered by the Nuclear Specification, it is permitted to base the adequacy of the designs on tests, analysis, or successful use, subject to the approval of the authority having jurisdiction.

> **User Note:** With the exception of hollow structural sections (HSS), for the design of structural members that are cold-formed to shapes with elements not more than 1 in. (25 mm) in thickness, the use of provisions of the ANSI/AISI S100 *North*

American Specification for the Design of Cold-Formed Steel Structural Members is recommended, incorporating the loads and load combinations delineated in Section NB2.

NA2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

Add the following:

Crane Manufacturers Association of America
CMAA-70 *Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes*, 2020

U.S. Nuclear Regulatory Commission
NUREG-0800 *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants*, March 2007

U.S. Code of Federal Regulations (CFR)
Title 10 of the *Code of Federal Regulations*, Part 50 (10CFR50), Appendix B 2019, and Appendix S, 2020
Title 10 of the *Code of Federal Regulations*, Part 830, Subpart A, Quality Assurance Requirements (to be used for Department of Energy Nuclear Facilities), 2020
Title 10 of the *Code of Federal Regulations*, Part 100 (10CFR100), Reactor Site Criteria, 2019

U.S. Department of Energy (DOE)
DOE Order O 414.1D, *Quality Assurance*, April 2011
Nuclear Energy Institute (NEI)
NEI 07-13, *Methodology for Performing Aircraft Impact Assessments for New Plant Designs*, Revision 8P, 2011

Add the following to (a) American Concrete Institute (ACI):

ACI 117-10 *Specification for Tolerances for Concrete Construction and Materials and Commentary*
ACI 117M-10 *Specification for Tolerances for Concrete Construction and Materials and Commentary (Metric)*

Add the following to (b) American Institute of Steel Construction (AISC):

ANSI/AISC 360-22 *Specification for Structural Steel Buildings*

Delete the following in (b) American Institute of Steel Construction (AISC):

ANSI/AISC N690-18 *Specification for Safety-Related Steel Structures for Nuclear Facilities*

Add the following to (c) American Society of Civil Engineers (ASCE)

ANSI/ASCE 8-22 *Specification for the Design of Cold-Formed Stainless Steel Structural Members*

Add the following to (d) American Society of Mechanical Engineers (ASME)

ASME NQA-1-2022 *Quality Assurance Requirements for Nuclear Facility Applications*, 2022

ASME *Boiler and Pressure Vessel Code* Section III, Div. 1, 2023

Add the following to (f) ASTM International (ASTM):

A106/A106M-19a *Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service*

A240/A240M-23 *Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications*

A276/A276M-17 *Standard Specification for Stainless Steel Bars and Shapes*

A312/A312M-22a *Standard Specification for Seamless, Welded, and Heavily Cold Worked Austenitic Stainless Steel Pipes*

A320/A320M-22a *Standard Specification for Alloy-Steel and Stainless Steel Bolting for Low-Temperature Service*

A479/A479M-23a *Standard Specification for Stainless Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels*

A515/A515M-17(2022) *Standard Specification for Pressure Vessel Plates, Carbon Steel, for Intermediate- and Higher-Temperature Service*

A516/A516M-17 *Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service*

A537/A537M-20 *Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel*

A540/A540M-15(2021) *Standard Specification for Alloy-Steel Bolting Materials for Special Applications*

A564/A564M-19a *Standard Specification for Hot-Rolled and Cold-Finished Age-Hardening Stainless Steel Bars and Shapes*

A666-15 *Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar*

A738/A738M-19 *Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel, for Moderate and Lower Temperature Service*

A1008/A1008M-21a *Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with*

Improved Formability, Required Hardness, Solution Hardened, and Bake Hardenable

Add the following to (g) American Welding Society (AWS)

AWS A5.4/A5.4M:2012(R2022) *Specification for Stainless Steel Electrodes for Shielded Metal Arc Welding*
AWS A5.9/A5.9M:2022 *Welding Consumables-Wire Electrodes, Strip Electrodes, Wires, and Rods for Arc Welding of Stainless and Heat Resisting Steels – Classification*
AWS A5.22/A5.22M:2012 *Specification for Stainless Steel Flux Cored and Metal Cored Welding Electrodes and Rods*
AWS D1.4/D1.4M:2018-AMD1 *Structural Welding Code— Steel Reinforcing Bars*
AWS D1.6/D1.6M:2017-AMD1 *Structural Welding Code—Stainless Steel*
AWS D1.8/D1.8M:2021 *Structural Welding Code—Seismic Supplement*

NA3. MATERIAL

1. Structural Steel Materials

Replace section with the following:

In addition to satisfying the applicable ASTM standards, the specification of the material of those structures or structural components that are subject to impactive and/or impulsive loads shall be supplemented by the requirement that the material be subjected to Charpy V-notch (CVN) impact tests, using the procedures described in ASTM A673/A673M. The CVN impact test shall be conducted at a temperature of 0°F (-18°C). For plates and structural shapes with plate thicknesses and flange thicknesses, respectively, equal to or less than 2 in. (50 cm) and for weld metal, the acceptance criteria shall be based on energy values indicated in Table NA3.1, in addition to satisfying the applicable ASTM and AWS standard.

User Note: Higher fracture toughness is available for certain materials not produced as rolled sections, but only available as plate or bar. Where the fracture toughness of materials available in rolled shapes does not meet the requirements of Table NA3.1 at 0°F (-18°C), the component may be fabricated from plate or bar provided all requirements (CVN and others) applicable to the fabricated shape are met.

User Note: For material strengths that exceed the requirements in this section, project-specific CVN requirements will need to be established.

TABLE NA3.1

Charpy V-Notch Energy Values		
Specified Minimum Yield Stress	Charpy V-Notch Energy Value	
	Average of Three Specimens, Minimum	One Individual Specimen, Minimum
36 ksi (250 MPa) up to and including 65 ksi (450 MPa)	25 ft-lb (34 J)	20 ft-lb (27 J)
Matching 70 ksi (480 MPa) and 80 ksi (550 MPa) weld filler metal	25 ft-lb (34 J)	20 ft-lb (27 J)

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 CVN requirements of Table NA3.1 and conforms with one of the ASTM standards listed in *Specification* Table A3.1, subject to the grades and limitations listed.

1a. Listed Materials

Modify Table A3.1 as follows:

(b) Hollow structural sections (HSS)

Add the following:

ASTM A106/A106M
ASTM A312/A312M

(c) Plates

Add the following:

ASTM A240/A240M
ASTM A515/A515M
ASTM A516/A516M
ASTM A537/A537M Class 1 and Class 2
ASTM A709/A709M
ASTM A738/A738M Grades B and C

(d) Bars

Add the following:

ASTM A276
ASTM A479/A479M

(e) Sheets

Add the following:

ASTM A666
ASTM A1008/A1008M

Add the following:

For the design of structural members cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steels, refer to Sections 3, 4, and 5 of ANSI/ASCE 8. ANSI/ASCE 8 is not applicable for hot-rolled or built-up steel members, assemblies, and connections.

User Note: For guidance regarding the design and fabrication of stainless steel members, assemblies and connections, refer to ANSI/AISC 370-21, *Specification for Structural Stainless Steel Buildings*, and AISC Design Guide 27, *Structural Stainless Steel.* Additional requirements for stainless steel plates used in steel-plate composite (SC) structural elements can be found in Appendix N9.

User Note: Weldability should be considered when selecting material to be used in welded applications, especially when selecting stainless steel.

User Note: Materials at the interface of SC elements and elements governed by ASME *Boiler and Pressure Vessel Code*, Section II, are to be procured using ASME SA grade designations rather than the corresponding ASTM designations.

1c. Unidentified Steel

Replace section with the following:

Unidentified steel shall not be used.

1d. Rolled Heavy Shapes

Add the following:

The design documents shall identify welded connections that are determined by the engineer of record to be susceptible to lamellar tearing. For such connections, a plan shall be developed by the engineer of record (EOR) to mitigate the conditions creating the potential for lamellar tearing.

User Note: In determining the need for prefabrication 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 3/4 in. (19 mm).

1e. Built-Up Heavy Shapes

Add the following:

The design documents shall identify welded connections that are determined by the engineer of record to be susceptible to lamellar tearing. For such connections, a plan shall be developed by the EOR to mitigate the conditions creating the potential for lamellar tearing.

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-90 (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 Plates 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.

3. Bolts, Washers and Nuts

(a) Bolts

Add the following:

ASTM A320/A320M
ASTM A540/A540M

ASTM A564/A564M

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 A5.23/A5.23M
AWS A5.4/ A5.4M	AWS A5.25/A5.25M
AWS A5.5/ A5.5M	AWS A5.26/A5.26M
AWS A5.9/ A5.9M	AWS A5.28/ A5.28M
AWS A5.17/A5.17M	AWS A5.29/ A5.29M
AWS A5.18/ A5.18M	AWS A5.32M/A5.32
AWS A5.20/A5.20M	
AWS A5.22/A5.22M	

CVN requirements are provided in Section NJ2.6.

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 typically 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.

Add the following section:

7. Material Certification

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 applicable specification.

NA4. STRUCTURAL DESIGN DOCUMENTS AND SPECIFICATIONS

1. Structural Design Documents and Specifications Issued for Construction

Add the following:

The structural design documents and specifications shall meet the following requirements:

Structural elements or systems with cyclic loads shall be so indicated as well as the number of cycles, when applicable. Additionally, structural elements or systems that are subject to impactive and/or impulsive loads shall be identified. The documents for the structural elements shall identify those elements or systems that are deemed safety-related by the engineer of record.

The structural design documents and specifications shall include:

(1) Applicable 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

(10) Identify CJP welds to be 100% inspected by either UT or RT.

Add the following section:

NA6. 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 shall be those established by 10CFR50, Appendix B (for Nuclear Power Stations) and in 10CFR830, Subpart A and DOE Order O 414.1D (for DOE Nuclear Facilities). Additional quality assurance requirements shall meet the requirements of Chapter NN.

Analysis and design calculations 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 authority having jurisdiction (AHJ). Activities involving specifications, analyses, designs, calculations, documentation, fabrication, and erection shall be 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, and 10CFR830, Subpart A, provide regulations for quality assurance (QA) and quality control (QC). Both these documents defer many requirements to ASME NQA-1. The requirements of Chapter NN are aimed to further assist the user in developing a QA/QC program that will satisfy these regulations for safety-related structural steel, composite, and steel-plate composite (SC) structures.

It is noted that the Nuclear Specification uses the term “safety-related” as being applicable to both commercial nuclear safety related structures as well as “safety-class” structures (as defined in the pertinent DOE documents). However, for both types of facilities, the engineer of record may elect to apply the associated design and quality assurance requirements to less safety-critical structures (e.g., certain important-to-safety or Risk Informed Safety Class structures in commercial nuclear power plants and safety-significant structures in DOE nuclear facilities).

CHAPTER NB

DESIGN REQUIREMENTS

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.

The chapter is organized as follows:

NB2. LOADS AND LOAD COMBINATIONS

Replace section with the following:

Safety-related steel structures for nuclear facilities shall be designed using the normal loads, severe environmental loads, extreme environmental loads, abnormal loads, and load combinations of this section.

1. Normal Loads

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

D = 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 = 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)
F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights
H = loads due to weight and pressure of soil, water in soil, or bulk materials
L = live load due to occupancy and moveable equipment, including impact
L_r = roof live load

R = rain load

R_o = pipe reactions during normal operating, start-up, or shutdown conditions, based on the most critical transient or steady-state condition

S = snow load

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

Snow loads shall be as stipulated in Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7) for Risk Category IV facilities.

2. Severe Environmental Loads

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

E_o = loads generated by the operating basis earthquake (OBE)

W = wind load

Operating basis earthquake loads shall be as defined in 10CFR50, Appendix S, or as specified by the AHJ. Wind loads shall be as stipulated in ASCE/SEI 7 for Risk Category IV facilities, or as specified by the AHJ.

User Note: The OBE is an 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.

3. Extreme Environmental Loads

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

E_s = loads generated by the safe shutdown earthquake (SSE), or design basis earthquake (DBE)

W_t = loads generated by the specified design (basis) tornado, including wind pressures, pressure differentials, and tornado-borne missiles

Safe shutdown earthquake loads shall be as defined in 10CFR50, Appendix S, or as specified by the AHJ. Tornado-based loads shall be as defined in the U.S. Nuclear

Regulatory Commission Standard Review Plan Section 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 = maximum differential pressure load generated by the postulated accident
R_a = pipe and equipment reactions generated by the postulated accident, including R_o
T_a = thermal loads generated by the postulated accident, including T_o
Y_j = jet impingement load generated by the postulated accident
Y_m = missile impact load, such as pipe whip generated by or during the postulated accident
Y_r = 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, ϕR_n, of each structural component shall be equal to or greater than the required strength, R_u, determined from the applicable critical combinations of the loads. The possibility of one or more loads not acting concurrently shall be considered when determining the load combination(s) that produce the most critical structural effects. The load combinations specified in this section shall be investigated.

User Note: The above provision regarding situations when one or more loads may not be acting concurrently is particularly relevant to various "abnormal loads" and the tornado load effects (i.e., for load combinations listed under Section NB2.5c). This is explained further in the Commentary.

5a. Normal Load Combinations

$$1.4(D + R_o + F) + T_o + C \tag{NB2-1}$$

$$1.2(D + R_o + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) + 1.2T_o + 1.4C \tag{NB2-2}$$

$$1.2(D + R_o + F) + 1.6(L_r \text{ or } S \text{ or } R) + 0.8(L + H) + 1.2T_o + 1.4C \tag{NB2-3}$$

5b. Severe Environmental Load Combinations

$$1.2(D + F + R_o) + W + 0.8L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R) + T_o + C \tag{NB2-4}$$

$$1.2(D + F + R_o) + 1.6E_o + 0.8L + 1.6H + 0.2(L_r \text{ or } S \text{ or } R) + T_o + C \tag{NB2-5}$$

5c. Extreme Environmental and Abnormal Load Combinations

$$D + 0.8L + C + T_o + R_o + E_s + F + H \tag{NB2-6}$$

$$D + 0.8L + T_o + R_o + W_t + F + H \tag{NB2-7}$$

$$D + 0.8L + C + 1.2P_a + R_a + T_a + F + H \tag{NB2-8}$$

$$D + 0.8L + \left(P_a + R_a + T_a\right) + \left(Y_r + Y_j + Y_m\right) + 0.7E_s + F + H \tag{NB2-9}$$

5d. Other Considerations

The following additional requirements shall be considered in the loads and load combinations:

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

User Note: The action of T_a can lead to large member forces due to external or internal restraints. An effective way to minimize the effect of T_a is to incorporate design features that help accommodate thermal deformations (e.g., by using connections with long-slotted holes in the direction of thermal movement, partially restrained connections, expansion joints). Structural analysis including design for T_a should account for the presence of such features. See the Commentary for additional guidance regarding analysis of load effects due to T_a including benefits of using the direct analysis method described in Chapter C of the *Specification*.

(2) Where the structural effect of differential settlement is significant, it shall be included with the soil pressure 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 provided the load does not contribute to the destabilizing effect. F shall be treated in the same manner as D, and H shall be treated in the same manner as L when stability evaluations are performed.

(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_a, R_a, T_a, 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 including concentrated loads, Y_j, Y_r, and Y_m, or tornado-borne missiles, local

section strength is permitted to 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 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 1×10^{-6}.

6. Allowable Strength Design (ASD)

The allowable strength, R_n/Ω, of each structural component shall be equal to or greater than the required strength, R_a, determined from the critical combinations of the loads. The possibility of one or more loads not acting concurrently shall be considered when determining the load combination(s) that produce the most critical structural effects. The load combinations specified in this section shall be investigated.

User Note: The above provision regarding situations when one or more loads may not be acting concurrently is particularly relevant to various "abnormal loads" and the tornado load effects (i.e., for load combinations listed under Section NB2.6c). This is explained further in the Commentary.

6a. Normal Load Combinations

$$D + L + R_o + F + H + T_o + C \qquad \text{(NB2-10)}$$

$$D + \left(L_r \text{ or } S \text{ or } R\right) + R_o + F + H + T_o + C \qquad \text{(NB2-11)}$$

$$D + F + 0.75L + 0.75H + 0.75\left(L_r \text{ or } S \text{ or } R\right) + T_o + C \qquad \text{(NB2-12)}$$

6b. Severe Environmental Load Combinations

$$D + R_o + F + 0.6W + 0.75(L + H) + C + 0.75(L_r \text{ or } S \text{ or } R) + T_o \qquad \text{(NB2-13)}$$

$$D + R_o + F + E_o + 0.75(L + H) + C + 0.75(L_r \text{ or } S \text{ or } R) + T_o \qquad \text{(NB2-14)}$$

6c. Extreme Environmental and Abnormal Load Combinations

$$D + L + C + R_o + T_o + E_s + F + H \qquad \text{(NB2-15)}$$

$$D + L + R_o + T_o + W_t + F + H \qquad \text{(NB2-16)}$$

$$D + L + C + P_a + R_a + T_a + F + H \qquad \text{(NB2-17)}$$

$$D + L + P_a + R_a + T_a + Y_r + Y_j + Y_m + 0.7E_s + F + H \qquad \text{(NB2-18)}$$

6d. Other Considerations

The following additional requirements shall be considered in the loads and load combinations:

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

> **User Note:** The action of T_a can lead to large member forces due to external or internal restraints. An effective way to minimize the effect of T_a is to incorporate design features that help accommodate thermal deformations (e.g., by using connections with long-slotted holes in the direction of thermal movement, partially restrained connections, expansion joints). Structural analysis including design for T_a should account for the presence of such features. See the Commentary for additional guidance regarding analysis of load effects due to T_a including benefits of using the direct analysis method described in Chapter C of the *Specification*.

(2) Where the structural effect of differential settlement is significant, it shall be included with the soil pressure 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 provided the load does not contribute to the destabilizing effect. F shall be treated in the same manner as D, and H shall be treated in the same manner as L when stability evaluations are performed.

(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_a, T_a, 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 including concentrated loads Y_j, Y_r, and Y_m or tornado-borne missiles, local section strength is permitted to 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 1×10^{-6}.

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.

2. 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).

3. Required Strength

***Replace section with the following*:**

The required strength of structural members and connections shall be determined by structural analysis for the applicable 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, Section N1.3, Design by Inelastic Analysis.

The yield stress, modulus of elasticity, and proportional limit of carbon steel shall be investigated and reduced, as appropriate, for temperatures in excess of 250°F (120°C).

8. Design for Serviceability

Add the following:

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

Add the following section:

14. Analysis, Design, and Detailing for Impulsive and Impactive Loads

The analysis, design, and detailing of structural steel, composite members, and SC structural elements subjected to impulsive and impactive loads shall be evaluated in accordance with Appendix N10.

NB5. FABRICATION AND ERECTION

Replace section with the following:

Fabrication documents, fabrication, shop painting, erection documents, 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 NA6, 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 shall conform to the requirements of Appendix N5, Evaluation of Existing Structures.

CHAPTER NC

DESIGN FOR STABILITY

Modify Chapter C of the Specification as follows.

Add the following to the end of the first paragraph of Section C1:

The effects of elevated temperatures on the stability of the structure and its elements shall be considered.

Replace the User Note in Section C2.2 with the following:

User Note: The imperfections required to be considered in this section are imperfections in the locations of points of intersection of members (system imperfections). In typical building structures, the system imperfection is the out-of-plumbness of columns. For structures that do not fit the construct of a typical building (e.g., structural elements supporting mechanical and electrical components), the notional loads defined in Section C2.2b of the *Specification* are not always applicable and initial imperfections should be applied per Section C2.2a. Consideration of initial out-of-straightness of individual members (member imperfections) is not required in the structural analysis when using the provisions of this section; it is accounted for in the compression member design provisions of Chapter E of the *Specification* and need not be considered explicitly in the analysis as long as it is within the limits specified in the *Code of Standard Practice*. *Specification* Appendix 1, Section 1.2, provides an extension to the direct analysis method that includes modeling of member imperfections (initial out-of-straightness) within the structural analysis.

Replace the User Note in Section C2.2a with the following:

User Note: Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections. The magnitude of the initial displacements should be based on permissible construction tolerances, as specified in the *Code of Standard Practice* or other governing requirements, or on actual imperfections, if known. The direct application of these imperfections is intended to contribute to the destabilizing effects of the loads, i.e., P-Δ and P-δ, but is not intended to directly contribute to the imposed stresses due to support displacements.

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

Add the following after the first paragraph of the preamble to Chapter I.

The applicability of the requirements for composite plate shear walls shall be limited to standalone shear walls.

User Note: Typical safety-related nuclear facilities involve a labyrinthine grid of squat, shear-controlled steel-plate composite (SC) walls. Such shear-controlled walls are to be designed per Appendix N9. However, in some situations, certain nuclear facilities may involve tall, flexure-controlled standalone SC walls. Such flexure-controlled walls are to be designed using the provisions of this Chapter.

Modify Chapter I of the Specification as follows.

In Section I1.1, replace "*Building Code Requirements for Structural Concrete and Commentary* (ACI 318) and the *Metric Building Code Requirements for Structural Concrete and Commentary* (ACI 318M)" ***with*** "*Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary* (ACI 349) and the *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Metric)* (ACI 349M)".

For all instances in Chapter I, replace "ACI 318" ***with*** "ACI 349 or ACI 349M" ***and replace*** "ACI 318 Chapter 17" ***with*** "ACI 349 or ACI 349M, Appendix D."

Delete the following from Section I.1.3(a): "and not less than 3 ksi (21 MPa) nor more than 6 ksi (41 MPa) for lightweight concrete."

Add the following to the end of Section I1.3(a): "Lightweight concrete shall not be used."

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. Welded Alterations to Structures with Existing Rivets or Bolts

The use of the combined strength of existing rivets or bolts and welds on a common faying surface shall not be permitted.

Replace Section J1.10 with the following:

10. High-Strength Bolts in Combination with Existing Rivets

The use of the combined strength of existing rivets and high-strength bolts on a common faying surface shall not be permitted.

NJ2. WELDS AND WELDED JOINTS

Modify section as follows.

6. Filler Metal Requirements

Replace second paragraph with the following:

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

(a) Complete-joint-penetration (CJP) groove welded T- and corner joints with steel backing left in place when the joint is subjected to tension normal to the effective area of the weld, unless the joint is designed using the available strength for a partial-joint-penetration groove weld.

(b) CJP groove welded splices subject to tension normal to the effective area in heavy sections as defined in *Specification* Sections A3.1d and A3.1e.

Welds subject to impactive and/or impulsive loads shall be made with filler metals meeting the requirements specified in AWS D1.8/D1.8M, clauses 6.1, 6.2, and 6.3.

NJ3. BOLTS, THREADED PARTS, AND BOLTED CONNECTIONS

Modify section as follows.

2. High-Strength Bolts

Add the following to paragraph (b):

(4) Connections for supports of running machinery, or of other live loads that produce impact or reversal of stress

(5) Other connections stipulated on the design documents

Add the following to paragraph (c):

(3) For supports of vibrating machinery and other situations where high-cycle fatigue is a design concern

User Note: See Appendix N3 for design of joints subject to high-cycle fatigue.

11. Bearing and Tearout Strength at Bolt Holes

Replace paragraph (a) of Section J3.11a(1) with the following:

(a) Bearing

$$R_n = 2.4dtF_u \quad \text{(J3-6a)}$$

Replace paragraph (b) of Section J3.11a(1) with the following:

(b) Tearout

$$R_n = 1.2l_ctF_u \quad \text{(J3-6c)}$$

Replace paragraph (i) of Section J3.11b(2) with the following:

(i) For a bolt in a connection with a standard hole or a short-slotted hole with the slot perpendicular to the direction of force

$$R_n = 1.2l_ctF_u \quad \text{(J3-6g)}$$

User Note: Deformation at bolt holes is always a design consideration in nuclear facilities.

Add the following new section:

14. Connections for Members Subject to Impactive or Impulsive Loads

Bolted connections for members that are subject to impactive or impulsive loads shall be configured such that a ductile limit state controls the connection design.

CHAPTER NK

ADDITIONAL REQUIREMENTS FOR HSS AND BOX-SECTION CONNECTIONS

No changes to Chapter K of the Specification.

CHAPTER NL

DESIGN FOR SERVICEABILITY

Modify Chapter L of the Specification as follows.

Replace preamble with the following:

This chapter addresses serviceability design requirements.

The chapter is organized as follows:

NL1. General Provisions
NL2. Deflections
NL3. Drift
NL4. Vibration
NL5. Wind-Induced Motion
NL6. Thermal Expansion and Contraction
NL7. Connection Slip

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 applicable load combinations stipulated in Section NB2 and the applicable Appendices.

User Note:

Reduced stiffness values used in the direct analysis method, described in Chapter C of the *Specification*, are not intended for use with the provisions of this chapter. However, Section NB3.8 does require that stiffness reduction due to elevated temperatures be considered for serviceability.

CHAPTER NM

FABRICATION AND ERECTION

Modify Chapter M of the Specification as follows.

NM1. FABRICATION AND ERECTION DOCUMENTS

Replace section with the following:

Fabrication and erection documents are permitted to be prepared in stages. Fabrication documents 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 documents shall be prepared in advance of erection and give information necessary for erection of the structure. Fabrication and erection documents shall clearly distinguish between shop and field welds and between shop and field 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.

Fabrication and erection documents 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 the lesser of the maximum specified in the applicable ASTM standard or 1,200°F (650°C) for carbon steels. For ASTM A514/A514M and ASTM A709/A709M Grade 70, the temperature of heated areas shall not exceed 1,100°F (590°C). The temperature of heated areas for ferritic, martensitic, or duplex stainless steels shall not exceed 600ºF (320°C). The temperature of heated areas for austenitic stainless steel shall not exceed 800ºF (430°C). The temperature of heated areas for precipitation hardening stainless steel shall not exceed the ageing temperature. Subject to the approval of the EOR, alternative temperature limitations are permitted to be used based on recommendations by the material producer.

2. Thermal Cutting

Modify first paragraph to read as follows:

Thermally cut edges shall meet the requirements of AWS D1.1/D1.1M, clauses 7.14.5.2 , 7.14.8.3, and 7.14.8.4 with the exception that thermally cut free edges that will not be subject to fatigue shall be free of sharp V-shaped notches and gouges greater than 3/16 in. (5 mm) in depth. Gouges deeper than 3/16 in. (5 mm) and notches shall be removed by grinding or repaired by welding. Notches or gouges deeper than 3/16 in. (5 mm) and up to 3/8 in. (10 mm) that remain from cutting shall be removed by grinding at a slope not greater than 1:2.5. Notches or gouges 3/8 in. (10 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:

Welding shall be performed in accordance with AWS D1.1/D1.1M and AWS D1.6/D1.6M except as modified in Section NJ2.

User Note: Welder qualification tests on plate defined in AWS D1.1/D1.1M, clause 10, and AWS D1.6/D1.6M, clause 6, are appropriate for welds connecting plates, shapes, or HSS to other plates, shapes, or rectangular HSS.

The 6GR tubular welder qualification shall be required for unbacked complete-joint-penetration groove welds of HSS T-, Y- and K-connections.

When the elements of a steel-plate composite (SC) structural element are welded to Class MC components in accordance with ASME *Boiler and Pressure Vessel Code,* Section III, Class NE, the requirements of Subsection NE shall govern the weld at the interface.

Weld on safety-related material shall be uniquely identified and shall be uniquely traceable.

User Note: Parameters documented and retrievable for each weld include, but are not limited to, the welder, weld wire lot/filler metal used, equipment used, date the weld was performed, date the weld was inspected, identification of weld inspector, and weld WPS used. The fabricator or constructor, as applicable for the work scope, should develop a method whereby each weld and its associated data can be identified.

7. Dimensional Tolerances

Replace section with the following:

Dimensional tolerances shall be in accordance with *Code of Standard Practice*, Section 11, and as listed in the following.

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

(a) 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 in length and 1/8 in. (3 mm) for members over 30 ft (9 m) in length.

In compression members, erection holes or holes mispunched or misdrilled are permitted to be left unfilled provided the net area is not less than 0.85 times the gross area. In tension members, holes are permitted to be left unfilled provided the net area requirements are met. In either condition, the unfilled holes shall not violate the minimum hole spacing requirements of *Specification* Section J3.4.

(b) Stiffeners

Stiffeners serving as connections shall be located within 1/4 in. (6 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 one-half of their thickness from the detailed position.

(c) 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.

(d) Steel-Plate Composite (SC) Structural Elements

Dimensional tolerances of SC structural elements as measured in the fabrication shop shall be as follows:

(1) At tie locations, the perpendicular distance between the opposite faceplates are within plus or minus t_{sc}/200, rounded upward to the nearest

1/16 in. (2 mm), where t_{sc} is the SC section thickness. This tolerance check shall be performed for the row of ties located closest to the free edges of SC panels.

(2) In between the tie locations, the perpendicular distance between the opposite faceplates are within plus or minus $t_{sc}/100$, rounded upward to the nearest 1/16 in. (2 mm). This tolerance check shall be performed along the free edges of the SC structural elements.

(3) The tie locations (tie spacing) conform to the shear connector provisions of AWS D1.1/D1.1M or AWS D1.6/D1.6M, as applicable.

(4) The squareness and the skewed alignment of opposite faceplates are such that the applicable dimensional tolerances for making the connections between adjacent panels, sub-modules, or modules are met. No additional squareness or skewed alignment tolerances are required.

User Note: Items (1) and (2) also define the tolerance for tie length relative to the SC section thickness. The tolerance for individual tie components (i.e., parts that make up the tie) should be based on the *Code of Standard Practice*, provided that the overall tolerance requirements (1) and (2) are satisfied.

Dimensional tolerances for fit-up of adjoining panels, sub-modules, or modules, as measured before making connections between faceplates of these panels, sub-modules, or modules shall be as follows:

(1) The fit-up tolerance of faceplates of adjoining SC structural elements, sub-modules, or modules joined together by welding shall be governed by the tolerances in AWS D1.1/D1.1M, AWS D1.4/D1.4M, or AWS D1.6/D1.6M, as applicable.

(2) The fit-up tolerance of faceplates of adjoining panels, sub-modules, or modules joined together by bolting shall be governed by the applicable requirements of the *Code of Standard Practice*.

User Note: These dimensional tolerances for fit-up of adjoining panels, sub-modules, or modules are to be checked before making the connections, i.e., at the fabrication yard or at the site, depending on the construction sequence. The engineer of record may specify additional dimensional tolerances in the contract documents for the fabrication of panels to achieve the dimensional tolerances for fit-up of faceplates of adjoining panels, sub-modules, or modules.

Before concrete is placed, the dimensional tolerances for erected modules shall be governed by the erection tolerances defined in the *Code of Standard Practice,* Section 7.13, with the exception that the working lines will be located at one faceplate of the SC structural element.

Dimensional tolerances for SC structural elements after concrete curing shall be governed by the concrete construction tolerances defined in ACI 349 or ACI 349M and ACI 117 or ACI 117M.

Additionally, after concrete curing, the faceplate waviness, f_w, shall be limited to the following:

$$f_w \le \left(\frac{t_p}{2}\right)\left(\frac{s_{t,min}}{s}\right) \tag{NM2-1}$$

where

s = spacing of the steel anchors, in. (mm)
$s_{t,min}$ = minimum tie spacing, in. (mm)
t_p = thickness of faceplate, in. (mm)

For cases where only ties are used, $s_{t,min}/s$ shall be taken as 1.0 in Equation NM2-1.

User Note: The engineer of record may specify the concrete pour rate and height to meet the faceplate waviness requirements.

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 materials shall not be less than that specified in ASTM A6, Table X4.1 and X4.2. The engineer of record shall provide the minimum bending radius for materials not listed in ASTM A6.

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 ASME NQA-1. The engineer of record shall provide the fabricator with the critical material characteristics based on the applicable ASTM or other national material or product standards 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 shall be identified in one of the following ways as defined by the required use of the material. The material’s use shall be defined by the contract documents. If the contract documents do not define the type of identification required, the identification defined in item (a) in the following shall control.

(a) 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 shall be obtainable throughout the service life of the structure. The fabricator shall maintain the documentation until such time that those documents are transferred to the owner.

(b) Material identified by heat number for the structure only. Material test reports shall be identifiable to the structure, but need not be identifiable to an individual member in the structure.

(c) 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.

(d) 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.

Fabricators shall transfer material test report to the owner for material identified by (b), (c), or (d) and remain obtainable throughout the service life of the structure by the owner.

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 is removable prior to erection or that has characteristics that make removal prior to erection unnecessary. This rust-inhibitive coating shall be approved by the engineer of record. This machine-finished surface requirement shall not apply to no-paint areas required for field welding. Corrosion in these no-paint areas for welding is permitted as long as the amount of corrosion is not detrimental to the design intent.

User Note: Paint (coatings) procurement, application, and inspection for a nuclear facility is subject to multiple codes, standards, and regulations that 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 structures shall be carried up true and plumb within the limits defined in the *Code of Standard Practice* Section 11 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 is subjected, including equipment and the operation of same. For composite steel/concrete structures, the required bracing shall 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

Crane column base lines shall be established as parallel lines and the column centerlines maintained within 1/8 in. (3 mm) of the theoretical distance.

7b. Tolerances for Crane Runway Girders

Horizontal sweep in crane runway girders shall not exceed 1/4 in. (6 mm) per 50 ft (15 m) length of girder spans. Camber shall not exceed 1/4 in. (6 mm) per 50 ft (15 m) of the girder span over that indicated on the design documents.

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 tolerances in Sections NM4.7a and NM4.7b 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-section beams), in no case shall the real eccentricity be greater than 3/4 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. As noted in Section NN6, steel-plate composite (SC) construction designed in accordance with Appendix N9 shall comply with applicable provisions (for the concrete and concrete reinforcing steel) of ACI 349 or ACI 349M for tests, materials, and construction requirements. This chapter does not address quality control or quality assurance for surface preparation or coatings.

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

User Note: The provisions of this chapter are pertinent to the activities performed by the fabricator, erector, and associated parties. Consult Section NA6 for activities related to calculations and design.

The chapter is organized as follows:

- NN1. General Provisions
- 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 QC and QA programs. Nondestructive examination (NDE) shall be performed by an individual, agency, or firm approved by the fabricator or erector responsible for QA.

User Note: The producers of materials manufactured in accordance with standard specifications referenced in Section NA3 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 completed in accordance with the established quality assurance program, the appropriate elements of the standard, the Nuclear Specification, and the construction documents. The quality assurance program shall be developed in accordance with the ASME standard NQA-1, *Quality Assurance Requirements for Nuclear Facility Applications*, or equivalent.

Material identification procedures shall comply with the requirements of the *Code of Standard Practice*, Section 6.1, except that the identification of material deemed safety-related shall 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.

Using the approved fabrication documents, the fabricator's quality assurance inspector (QAI) shall perform inspections of 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 11 of the *Code of Standard Practice* and Chapter NM

User Note: The QAI may be employed by the EOR, detailer, fabricator, erector, contractor, and/or constructor.

Using the approved erection documents, the erector's QAI shall perform inspections of the following as a minimum, as applicable:

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

(2) Steel deck and steel headed 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 11 of the *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 and approval by the owner or the engineer of record or their designee in accordance with Section 4 of the *Code of Standard Practice*, prior to fabrication or erection, as applicable:

(1) Fabrication approval documents, unless fabrication documents have been furnished by the owner or the engineer of record

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

At completion of fabrication, the 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 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 *Specification* Section A3.2.

(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 *Specification* Section A3.4.

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

(7) For steel 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 erectors' QC and QA personnel 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 in compliance with the fabricator's or erector's quality assurance (QA) program.

User Note: The qualification requirements for the fabricator's or erector's inspectors will require review and approval by the owner or their designated representative. The QCI may be employed by the fabricator, erector, contractor, and/or constructor.

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, clause 8.1.4, and AWS D1.6, clause 6, if applicable to stainless steel welding.

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

NDE personnel shall be qualified in accordance with their employer's written practice, which shall meet the criteria of AWS D1.1/D1.1M, clause 8.1.4.2(4), and AWS D1.6, clause 8.1.4.2, if applicable to stainless steel welding, and

(a) American Society for Nondestructive Testing (ASNT) SNT-TC-1A*, Recommended Practice for the Qualification and Certification of Nondestructive Testing Personnel*, or
(b) ASNT CP-189, *Standard for the Qualification and Certification of Nondestructive Testing Personnel*.

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

For QC inspection, the applicable contract documents shall be the approval documents, and the applicable referenced specifications, codes, and standards.

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

2. Quality Assurance

Quality assurance (QA) inspection of fabricated items shall be made at the fabricator's plant.

QA inspection of the erected steel system shall be made at the project site.

User Note: The quality assurance inspection required on safety-related work is performed by an inspector employed by or contracted to the fabricator or erector. The fabricator or erector coordinates the work of the quality assurance inspector internally to meet the requirements of the project specifications, the Nuclear Specification, and the fabricator's or erector's quality program. Because this work is internal to the fabricator or inspector, it is typically their responsibility to coordinate the inspection tasks in such a manner as to minimize disruption of the work being performed.

Surveillance performed by the owner or the owner's representative is typically identified as witness or hold points in the design documents. In order to minimize work interruption, advance notice of the schedule for these witness or hold points should be identified in the specifications or design documents.

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 fabricated members and components are 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 listed for QA in Tables NN5.4-1 through NN5.4-3 and Tables NN5.6-1 through NN5.6-3 shall be 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 shall be the approval documents, specifications, and applicable reference codes and standards.

3. Coordinated Inspection

Where a task is to be performed by both QC and QA, it is permitted to coordinate the inspection function between the personnel qualified for QCI 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 shall 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. Applicable provisions of AWS D1.1/D1.1M, AWS D1.6/D1.6M, or AWS D1.3/D1.3M shall apply to all structural and stainless steel.

User Note: The technique, workmanship, appearance, and quality of welded construction are addressed in Section NM2.4.

User Note: Visual weld acceptance criteria can also be found in the Electric Power Research Institute document NCIG-01, Revision 2, "Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants," NP-5380, Volume 1, September 1987. These nonmandatory inspection guidelines may be used for visual inspection of structural welds made in accordance with the provisions of AWS D1.1/D1.1M if approved by the engineer of record. These guidelines provide background information and instructions to assist the inspector in evaluating weld attributes. Measuring techniques and guidance on the accuracy, frequency, and locations for measuring welds are discussed. It is important for the inspector to understand weld size tolerance and significant measurements units in order to properly assess the acceptance of each weld.

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 shall be as follows:

Observe (O)—The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.
Perform (P)—These tasks shall be performed for each welded joint or member.

TABLE NN5.4-1
Inspection Tasks Prior to Welding

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

[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

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

N/A = not applicable

TABLE NN5.4-3
Inspection Tasks After Welding

Inspection Tasks After Welding	QC	QA
Welds cleaned	P	O
Size, length, and location of welds	P	O
Welds meet visual acceptance criteria • Crack prohibition • Weld/base-metal fusion • Crater cross section • Weld profiles • Weld size • Undercut • Porosity	P	O
Arc strikes	P	O
k-area[1]	P	O
Backing removed and weld tabs removed (if required)	P	O
Repair activities	P	P
Document acceptance or rejection of welded joint or member	P	O

[1] 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.

User Note: The technique, workmanship, appearance, and quality of welded construction is addressed in Section NM2.4.

5b. CJP and PJP Groove Weld NDE

Where identified on the contract documents, complete-joint-penetration (CJP) groove-welded joints subjected to transversely applied tension loading in butt, T-, and corner joints, in materials 5/16 in. (8 mm) thick or greater, shall receive 100% UT or RT examination.

User Note: Many joints in design-basis accident situations undergo transversely applied tension. The EOR, when evaluating welded joints subject to 100% UT or RT examination, should determine the welded joints critical to the safe shutdown of a nuclear facility and convey this inspection requirement to the fabricator and erector. The intent of this requirement is not to establish that all welds that could undergo transversely applied tension be 100% inspected, but rather only the welds depended on for a safe shutdown.

As a minimum, all CJP welds shall be 10% inspected by UT or RT examination.

As a minimum, 10% of partial-joint-penetration (PJP) welds shall be inspected by MT or PT examination.

In lieu of performing 10% examinations on each CJP or PJP weld, the fabricator or erector is permitted to inspect 100% of one weld in 10 from a series of welds grouped in a population. Populations shall be established based on like thickness, materials, welded joint geometry, and welding processes to satisfy a minimum of 10% NDE inspections of CJP or PJP groove-welded joints. Final determination of this method shall be accepted by the EOR prior to the start of fabrication or construction.

User Note: The fabricator, erector, and EOR should identify, prior to construction, a method of quantifying the inspection requirements of groove welds. The intent of inspecting 100% of one weld in 10 in lieu of 10% of each welded joint is for the EOR, fabricator, and erector to determine the best approach to satisfy that inspections were performed but also minimize the impact to productivity, cost, and schedule, while maintaining the same level of safety that inspecting 10% of each weld accomplishes. As an example, populations can be established either by part number, drawing, WPS, work package, elevation, or by other means that identify the size of the weld population from which the 100%-of-one-weld-in-10 sample is selected; selections based off an individual welder is not advised. Testing should be a continuous process throughout fabrication and erection. Populations and testing need not carry over from fabricator to erector as the method of establishing the population may differ. The method of selecting the weld population and 10% sample should be reviewed and agreed upon by the engineer of record.

5c. Welded Joints Subjected to Fatigue

CJP groove welded joints subjected to fatigue shall be identified on the contract documents and be 100% inspected by either UT or RT.

5d. Increase in Rate of Groove Weld NDE

Groove weld NDE shall increase in the event of a weld rejection in accordance with the following:

(a) Populations inspected at 10%: An additional 10% section of the same welded joint shall be inspected. If NDE results determine the additional 10% section of the weld joint is acceptable, the remaining weld joints within the population shall remain at a 10% NDE inspection rate; if NDE results determine the additional 10% section of the weld joint is unacceptable, all weld joints within the population shall be inspected at a 100% rate.

(b) Populations inspected at 100% of one weld in 10: An additional weld joint within the same population shall be selected and 100% of the joint length shall be inspected. If NDE results determine the additional weld joint is acceptable, the remaining weld joints within the population shall remain at a 100% NDE inspection of one weld in 10; if NDE results determine the additional weld joint is unacceptable, all weld joints within the population shall be inspected at a 100% rate.

Increased groove weld NDE shall only be applicable to a single population. Extending increased groove weld NDE between populations shall not be permitted.

5e. 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.

When a weld is rejected on the basis of NDE, the NDE record shall indicate the location of the defect and the basis of rejection.

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 shall not be 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 shall be as follows:

Observe (O)—The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.
Perform (P)—These tasks shall be performed for each bolted connection.

TABLE NN5.6-1
Inspection Tasks Prior to Bolting

Inspection Tasks Prior to Bolting	QC	QA
Manufacturer's certifications available for fastener materials	N/A	P
Fasteners marked in accordance with ASTM requirements	P	O
Correct fasteners selected for the joint detail (grade, type, bolt length if threads are to be excluded from shear plane)	P	O
Correct bolting procedure selected for joint detail	P	O
Connecting elements, including the specified faying surface condition and hole preparation, if specified, meet applicable requirements	P	O
Pre-installation verification testing by installation personnel observed and documented for fastener assemblies and methods used (reference RCSC *Specification*, Section 7)	P	O
Correct storage provided for bolts, nuts, washers, and other fastener components (reference RCSC *Specification*, Section 2.2)	O	O
N/A = not applicable		

TABLE NN5.6-2
Inspection Tasks During Bolting

Inspection Tasks During Bolting	QC	QA
Fastener assemblies placed in all holes and washers (if required) are positioned as required	P	O
Joint brought to the snug-tight condition prior to the pretensioning operation	P	O
Fastener component not turned by the wrench prevented from rotating	P	O
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	P	O

Table NN5.6-3
Inspection Tasks After Bolting

Inspection Tasks after Bolting	QC	QA
Document acceptance or rejection of bolted connections	P	O

7. Inspection of Galvanized Structural Steel Main Members

Exposed cut surfaces of galvanized structural steel main members and exposed corners of rectangular HSS shall be visually inspected for cracks subsequent to galvanizing. Cracks shall be repaired or the member shall be rejected.

User Note: It is normal practice for fabricated steel that requires hot dip galvanizing to be delivered to the galvanizer and then shipped to the jobsite. As a result, inspection at the jobsite is common.

8. Other Inspection Tasks

The fabricator's QAI shall inspect the fabricated steel to verify compliance with the details shown on the approved fabrication documents.

User Note: This includes such items as correct application of shop joint details at each connection.

The erector's QAI shall inspect the erected steel frame to verify compliance with the details shown on the approved erection documents.

User Note: This includes such items as braces, stiffeners, member locations, and correct 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 and documented prior to placement of concrete.

The QAI shall inspect the fabricated steel or erected steel frame, as applicable, to verify compliance with the details shown on the construction documents.

User Note: This includes such items as braces, stiffeners, member locations, and correct application of field 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 section.

For welding of steel headed stud anchors, the provisions of AWS D1.1/D1.1M shall 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 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 shall be as follows:

P—Perform these tasks for each steel element.

For welding of faceplates, 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. Steel-plate composite (SC) structural element welding inspection of the module 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. Tests, materials, and construction requirements for concrete shall comply with the applicable provisions of ACI 349 or ACI 349M. In Tables NN6.2 and NN6.3, the inspection tasks are as follows:

P—Perform these tasks for each steel element.

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. 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, dispositioned as "use as is," or made suitable for its intended purpose as determined by the engineer of record.

Nonconformance reports shall remain open until a 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.

TABLE NN6.1
Inspection of Steel Elements of Composite Construction Prior to Concrete Placement

Inspection of Steel Elements of Composite Construction Prior to Concrete Placement	QC	QA
Verify placement and installation of steel deck and all deck accessories with construction documents	P	P
Verify size and location of welds, including support, sidelap, and perimeter welds	P	P
Verify welds meet visual acceptance criteria	P	P
Verify repair activities of decking and accessories, if applicable	P	P
Verify placement and installation of steel headed stud anchors: Check spacing, type, and installation	P	P
Verify repair activities of steel headed stud anchors, if applicable	P	P
Document acceptance or rejection of steel elements	P	P

TABLE NN6.2
Inspection of SC Structural Element Prior to Concrete Placement

Inspection of Steel Elements of Composite Construction Prior to Concrete Placement	QC	QA
Inspection of faceplates	P	P
Placement and installation of ties	P	P
Placement and installation of shear connectors	P	P
Document acceptance or rejection of steel elements	P	P

TABLE NN6.3
Inspection of SC Structural Element After Concrete Placement

Inspection of Steel Elements of Composite Construction after Concrete Placement	QC	QA
Inspection of faceplates	P	P
Document acceptance or rejection of steel elements	P	P

APPENDIX N1

DESIGN BY ADVANCED ANALYSIS

Modify Appendix 1 of the Specification as follows.

N1.3. DESIGN BY INELASTIC ANALYSIS

1. General Requirements

Add the following to the end of the first paragraph:

It is permitted to have localized inelastic behavior due to thermally induced load effects only in individual beams or their connections provided that an inelastic analysis of the associated structure demonstrates that the structure is able to maintain its global stability and structural integrity to withstand all other concurrently acting loads.

User Note: Unlike impulsive and impactive loads, which affect a single or a few structural members, the accident temperature load case generally affects a large portion, if not the entirety of a structure. Also, unlike the case of design for impulsive and impactive loads, where the affected members are a priori known and therefore selectively targeted for detailing in accordance with the requirements of Section NB3.14, the same approach is difficult to implement for the accident temperature load case (except for incorporating thermal-load relieving features mentioned in the User Note for Sections NB2.5 and NB2.6). Accordingly, only localized inelastic response in individual beams is permitted as long as it will not adversely affect the structure's ability to resist other loads (e.g., sustained gravity load and the design basis earthquake load, which are part of the governing extreme environmental and abnormal load combinations).

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 OF FILLED COMPOSITE MEMBERS (HIGH STRENGTH)

No changes to Appendix 2 of the Specification.

APPENDIX N3

FATIGUE

No changes to Appendix 3 of the Specification.

APPENDIX N4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

Modify Appendix 4 of the Specification as follows.

N4.1. GENERAL PROVISIONS

Add the following paragraphs after the introductory paragraph:

The intended functions of the structure under a design basis fire shall be stated in the design basis documents. The provisions of Appendix N4 shall be for life safety associated with evacuation of building occupants in the event of a design-basis fire. The Nuclear 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 evaluation for fire conditions, design material parameters at elevated temperatures during the design-basis fire event shall be those defined in *Specification* Table A-4.2.1 and Table NA-4.2.2. Other material parameter values are permitted to 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

3a. Thermal Elongation

Replace section with the following:

The coefficients of thermal expansion shall be taken as follows:

(a) For structural and reinforcing steels: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion shall be 7.8 x 10^{-6}/°F (1.4 x 10^{-5}/°C).

(b) For normal weight concrete: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion shall be 5.5 x 10^{-6}/°F (9.9 x 10^{-6}/°C).

User Note: Table A-4.2.1 in the *Specification* 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.

User Note: At 1,000°F (540°C), concrete starts to deteriorate rapidly and the strength of reinforcing steel will be affected. This should be taken into account in the design.

Replace Table A-4.2.2 with the following (delete reference to lightweight concrete):

TABLE NA-4.2.2
Properties of Concrete at Elevated Temperatures

Concrete Temperature °F (°C)	$k_c = f'_c(T) / f'_c$	$k_{Ec} = E_c(T)/E_c$	$\varepsilon_{cu}(T)$, %
	Normal Weight Concrete		Normal Weight Concrete
68 (20)	1.00	1.00	0.25
200 (93)	0.95	0.93	0.34
400 (200)	0.90	0.75	0.46
550 (290)	0.86	0.61	0.58
600 (320)	0.83	0.57	0.62
800 (430)	0.71	0.38	0.80
1000 (540)	0.54	0.20	1.06
1200 (650)	0.38	0.092	1.32
1400 (760)	0.21	0.073	1.43
1600 (870)	0.10	0.055	1.49
1800 (980)	0.05	0.036	1.50
2000 (1100)	0.01	0.018	1.50
2200 (1200)	0.00	0.00	0.00

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 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 (EOR) 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. Section N5.4 is only applicable to static vertical gravity loads applied to existing roofs or floors.

User Note: The scope of Appendix N5 follows the *Specification*. Where the evaluation is for existing safety-related structures subjected to other than static loads or load combinations, or where the evaluation uses dynamic load analysis, dynamic testing, or load tests other than those in the scope of Section N5.4, the EOR is responsible to show that the test and analytical evaluation methods employed are acceptable to the authority having jurisdiction (AHJ).

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 EOR 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 EOR 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. Steel grade shall be verified by either 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 ASTM A568/A568M, as applicable. Evidence shall exist that the material used was dedicated and traceability was maintained during fabrication and erection. When steel grade cannot be established by existing documentation, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure to establish the steel properties. Nominal steel properties of steel grades shall be used in the evaluation of existing structures by structural analysis. Use of steel tensile properties greater than nominal values is permissible only when it can be shown that (a) the coupons taken for CMTR or certified report represent the structure being evaluated, and (b) the value selected is derived from a statistical analysis indicating a 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 EOR.

User Note: Steel properties if established from a statistical analysis with a 95% or greater confidence level are generally considered to be conservative and acceptable. However, in nuclear facilities, the use of the actual properties from CMTR, certified report, and the results of tensile tests is generally not permitted by the AHJ.

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 Sections NA3.1d and NA3.1e are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the provisions of Section NA3.1e. If the notch toughness so determined does not meet the provisions of Section NA3.1e, the EOR shall determine if remedial actions are required.

5. Weld Metal

When specified by the EOR, representative samples of weld metal shall be obtained. The EOR 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 identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606/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.

In addition to Appendix N5, the available strength of members and connections shall be determined from applicable provisions of the Nuclear Specification chapters and appendices.

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 EOR'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. 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 does not increase by more than 10% above that at the beginning of the holding period. 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. 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. The service test load shall be held for a period of one hour, and deformations shall be recorded at the beginning and at the end of the one-hour holding period.

N5.5. EVALUATION REPORT

After the evaluation of an existing structure has been completed, the EOR 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 documents, 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

MEMBER STABILITY BRACING

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 ANALYSIS

No changes to Appendix 8 of the Specification.

Add the following Appendix.

APPENDIX N9

STEEL-PLATE COMPOSITE (SC) STRUCTURAL ELEMENTS

This appendix addresses the design and detailing requirements, including for seismic applications, for steel-plate composite (SC) structural elements and their connections. SC structural elements include SC walls, SC slabs, and SC basemats.

The SC structural elements consist of two steel faceplates that are connected to each other using ties. These faceplates act compositely with the concrete infill by means of shear connectors.

User Note: Composite plate shear walls in Chapter NI are similar to steel-plate composite (SC) walls. However, the requirements of Chapter NI are limited to standalone *shear walls*.

The appendix is organized as follows:

N9.1. Design Requirements
N9.2. Analysis Requirements
N9.3. Design of SC Structural Elements
N9.4. Design of SC Structural Element Connections

User Note: A flowchart to facilitate the use of the appendix has been provided in the Commentary.

N9.1. DESIGN REQUIREMENTS

1. General Provisions

The following provisions shall apply to SC structural elements:

(a) For exterior SC structural elements, the minimum section thickness, t_{sc} , shall be 15 in. (380 mm). For interior SC structural elements, the minimum t_{sc} shall be 10 in. (250 mm).

(b) Faceplates shall have a thickness, t_p, not less than 0.25 in. (6 mm) nor more than 1.5 in. (38 mm).

(c) The reinforcement ratio, ρ, shall have a minimum value of 0.015 and a maximum value of 0.10, where ρ is determined as follows:

$$\rho = \frac{2t_p}{t_{sc}} \quad \text{(A-N9-1)}$$

where

t_p = thickness of faceplate, in. (mm)

t_{sc} = SC section thickness, in. (mm)

(d) The specified minimum yield stress of faceplates, F_y, shall not be less than 50 ksi (350 MPa) nor more than 80 ksi (550 MPa).The minimum elongation shall be at least 15%, and the minimum tensile-to-yield ratio, F_u/F_y, shall be 1.20.

(e) The specified compressive strength of the concrete, f'_c, shall not be less than the greater of 4 ksi (28 MPa) or [0.04+0.80ρ] times F_y , nor more than 10 ksi (70 MPa).

Lightweight concrete shall not be used.

(f) The faceplates of SC structural elements shall be nonslender, as specified in Section N9.1.3.

(g) Composite action shall be provided between faceplates and concrete using shear connectors, in accordance with Section N9.1.4.

(h) The opposite faceplates shall be tied to each other, in accordance with the tie requirements specified in Section N9.1.5.

(i) For faceplates with holes, the nominal rupture strength per unit width, F_uA_{sn}, shall be greater than 1.10 times the nominal yield strength per unit width, F_yA_s,

where

A_s = gross area of the faceplates per unit width, in.^{2}/ft (mm^2/m)

A_{sn} = net area of the faceplates per unit width, in.^{2}/ft (mm^2/m)

User Note: The term faceplates with holes, used here, refers to faceplates that use tie configurations that involve threaded parts, which warrant the use of holes in faceplates to secure the tie and faceplate together. This is to be differentiated from the case where faceplates have openings or penetrations.

(j) Both faceplates shall have the same nominal thickness, t_p, and specified minimum yield stress, F_y.

(k) Steel ribs, if used, shall be embedded into the concrete no more than the lesser of 6 in. (150 mm) or the embedment depth of the steel headed stud anchor minus 2 in. (50 mm). The ribs shall be welded to the faceplates and anchored in the concrete to develop the full yield strength of their directly connected elements.

(l) Splices at the seams between adjoining faceplates shall be designed to develop the nominal yield strength of the weaker of two connected faceplates.

2. Design Basis

For design purposes, SC structural elements shall be divided into an interior region and connection regions. The connection regions shall consist of perimeter strips with a width not less than the SC section thickness, t_{sc}, and not more than twice the SC section thickness, $2t_{sc}$.

2a. Required Strength

The required strength for SC structural elements and their connections shall be determined through an elastic finite element analysis for the applicable load combinations, except as stated in Section N10.3.4.

User Note: As discussed in Section N10.3.4, a nonlinear inelastic dynamic analysis may be needed to determine the response of structures to impactive or impulsive loads.

2b. Design for Stability

Second-order analyses of structures with vertical SC structural elements need not be performed if the conditions of ACI 318 or ACI 318M, Section 6.2.5, are satisfied. Second-order effects shall be considered if the conditions of ACI 318 or ACI 318M, Section 6.2.5.1, are not satisfied.

3. Faceplate Slenderness Requirement

Faceplates shall be anchored to concrete using shear connectors. The width-to-thickness ratio of the faceplates, b/t_p, shall be limited as follows:

For connection regions,

$$\frac{b}{t_p} \leq 1.0\sqrt{\frac{E_s}{F_y}} \qquad \text{(A-N9-2a)}$$

For interior regions,

$$\frac{b}{t_p} \leq 1.20\sqrt{\frac{E_s}{F_y}} \qquad \text{(A-N9-2b)}$$

where

E_s = modulus of elasticity of steel
= 29,000 ksi (200 000 MPa) for carbon steel and duplex stainless steel
= 28,000 ksi (193 000 MPa) for austenitic stainless steel

F_y = specified minimum yield stress of faceplate, ksi (MPa)

b = largest unsupported length of the faceplate between rows of shear connectors, in. (mm)
t_p = thickness of faceplate, in. (mm)

4. Requirements for Composite Action

4a. Classification of Shear Connectors

Shear connectors with interfacial slip of at least 0.20 in. (5 mm), while maintaining an available strength greater than 90% of the peak shear strength, shall be classified as yielding shear connectors. Shear connectors not meeting this requirement shall be classified as nonyielding shear connectors.

User Note: The above requirements, which are somewhat different than the requirements in Section I8.4 of the *Specification*, are appropriate and adequate for SC structural elements. This is because, unlike composite beams, SC structural elements are two-dimensional, and at approximately $2t_{sc}$, their associated development length is typically a lot smaller than half of a composite beam span (i.e. the typical development length for a composite beam).

Steel headed stud anchors shall be classified as yielding shear connectors, and the available shear strength, Q_{cv}, shall be obtained using the *Specification*. Classification and available strength, Q_{cv}, for all other types of shear connectors shall be established through testing.

User Note: Ties, ribs, and steel headed stud anchors serve as shear connectors that enable composite action. The requirements for steel headed stud anchors, which are a yielding type shear connector, are provided in *Specification* Sections I8.1 and I8.3.

Where a combination of yielding and nonyielding shear connectors is used, the resulting shear connector system shall be classified as nonyielding. In these cases, the strength of yielding shear connectors shall be taken as the strength corresponding to the displacement at which the nonyielding shear connectors reach their ultimate strength.

4b. Spacing of Shear Connectors

Adjacent shear connectors shall be spaced not to exceed the minimum of the following:

(a) The spacing required to develop the yield strength of the faceplates over the development length, L_d, given as

$$s \le c_1 \sqrt{\frac{Q_{cv}^{avg} L_d}{T_p}} \qquad \text{(A-N9-3)}$$

where

L_d = development length, in. (mm)
$\le 3t_{sc}$

Q_{cv}^{avg} = weighted average of the available interfacial shear strengths of shear connectors, kips (N)

T_p = $F_y t_p$ (LRFD), kip/in. (N/mm)
= $F_y t_p /1.5$ (ASD), kip/in. (N/mm)

c_1 = 1.0 for yielding shear connectors
= 0.7 for nonyielding shear connectors

User Note: The Q_{cv}^{avg} concept and its determination is illustrated in the Commentary for Section N9.3.6a(a).

(b) The spacing required to prevent interfacial shear failure before out-of-plane shear failure of the SC section, given as

$$\cancel{s \le c_1 \sqrt{\frac{Q_{cv}^{avg} l (0.9 t_{sc})}{\frac{M_n}{2.5 t_{sc}}}}}\ s \le c_1 \sqrt{\frac{Q_{cv}^{avg} l t_{sc}}{\frac{M_n}{2.5 t_{sc}}}} \qquad \text{(A-N9-4)}$$

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where

M_n = nominal flexural strength per unit width of SC structural element, as defined in Section N9.3.3, kip-in./ft (N-mm/m)

l = 12 in./ft (1000 mm/m)

t_{sc} = SC section thickness, in. (mm)

User Note: Shear connector spacing will typically be governed by the requirement for the development length to be no more than three times the SC section thickness ($3t_{sc}$). However, for portions of the SC structure subjected to an extremely large out-of-plane moment gradient, the shear connector spacing is designed to achieve interfacial shear strength to be greater than $(M_n/2.5t_{sc})/(\cancel{0.9}t_{sc})$, which is a reasonable upper bound on interfacial shear demand because flexural behavior controls (in other words, because the shear span-to-depth ratio is greater than 2.5). See the Commentary for further explanation as well as for discussion of situations when the shear span-to-depth ratio is smaller than 2.5.

5. Tie Requirements

The opposite faceplates of SC structural elements shall be connected to each other using ties consisting of individual components such as structural shapes, frames, or bars.

User Note: Ties serve multiple purposes during empty module and service configurations of an SC structural element. The ties need to provide adequate strength and stiffness to empty modules during rigging/handling, transportation, and concrete placement operation. In the service condition, the ties provide structural integrity by enabling composite action, they prevent section splitting, and they serve as out-of-plane shear reinforcement. The out-of-plane shear strength contribution of the ties depends on the classification and spacing of the ties.

5a. Classification of Ties

Ties shall be classified as yielding shear reinforcement when

$$F_{ny} \leq 0.85 F_{nr} \tag{A-N9-5}$$

where

F_{nr} = nominal rupture strength of the tie, or the nominal strength of the associated welded or threaded connection, whichever is smaller, kips (N)

F_{ny} = nominal yield strength of the tie based on its gross area if no threads are present, or on its root area if it is threaded, kips (N)

Otherwise, ties shall be classified as nonyielding shear reinforcement.

User Note: For a tie with a stud welded connection to one of the faceplates conforming to AWS D1.1/D1.1M, the above check needs to be exercised only for the tie connection to the opposite faceplate.

5b. Tie Spacing

The tie spacing shall not exceed 1.0 times the section thickness, t_{sc}. The tie spacing-to-faceplate thickness ratio, s_{tl}/t_p or s_{tt}/t_p, shall be limited as follows:

$$\frac{s_{tl}}{t_p} \; or \; \frac{s_{tt}}{t_p} \leq 1.0\sqrt{\frac{E_s}{2\alpha+1}} \tag{A-N9-6}$$

$$\frac{s_{tl}}{t_p} \; or \; \frac{s_{tt}}{t_p} \leq 0.38\sqrt{\frac{E_s}{2\alpha+1}} \tag{A-N9-6M}$$

where

s_{tl} = spacing of ties in the longitudinal direction, in. (mm)

s_{tt} = spacing of ties in the transverse direction , in. (mm)

t_p = thickness of the faceplate, in. (mm)

t_{sc} = SC section thickness, in. (mm)

α = $1.7\left[\frac{t_{sc}}{t_p} - 2\right]\left[\frac{t_p}{D_{tie}}\right]^4$

D_{tie} = Equivalent diameter of shear reinforcement, in. (mm)

> **User Note:** A tie may be a circular structural element (e.g., tie rod) or an assembly of several structural elements (e.g., tie bar with gusset plate at one or both ends). The effective diameter of non-round ties will be direction (orientation) dependent. For a noncircular structural element, its cross-sectional area, A_{tie}, can be used to calculate $D_{tie} = \sqrt{\frac{4A_{tie}}{\pi}}$.

6. Design and Detailing Requirements for Impactive and Impulsive Loads

The analysis, design, and detailing of SC structural elements subject to impulsive and impactive loads shall be evaluated in accordance with Appendix N10.

7. Design and Detailing Requirements for Openings

> **User Note:** Faceplate holes for reinforcing steel dowels or for other types of joining instruments that are less than 2-1/2 in. (63 mm) in diameter, and $t_{sc}/8$, do not constitute as openings.

7a. Design and Detailing Requirements for Small Openings

All openings other than those classified as large openings shall be treated as small openings. It is permitted to neglect the effect of small openings where the largest dimension is equal to or less than 6 in. (150 mm) and not exceeding 25% of the SC structural element thickness provided that Section N9.1.7a(b) detailing requirements (1), (2), and (3) are satisfied.

The following requirements apply to small openings with the largest dimension greater than the lesser of (1) 6 in. (150 mm) and (2) 25% of the SC structural element thickness.

At the boundary of small openings, detailing shall be provided to achieve either a free edge or a fully developed SC structural element. Openings with free-edge detailing at their boundary are permitted only within the interior regions. Design and detailing shall be as follows:

(a) Design and detailing with a free edge at the perimeter of small openings

(1) Analysis is permitted to be performed without modeling the opening provided that the panel section where the opening is located shall be evaluated considering 25% reduction in all available strengths. Alternatively, the effect of a small opening shall be accounted for by

conducting an analysis that meets the Section N9.1.7b(a) requirements (1) and (2).

(2) Reentrant corners of noncircular or non-oval openings shall have corner radii not less than four times the faceplate thickness.

(3) The first row of ties around the opening shall be located at a distance from the opening no greater than one-quarter of the SC section thickness, t_{sc}.

(b) Design and detailing with fully developed edge at the perimeter of small openings

Sections surrounding the opening are permitted to be designed using the required strength based on an analysis model that does not consider the opening, provided the following detailing requirements are satisfied:

(1) Reentrant corners of noncircular or non-oval openings shall have corner radii not less than four times the faceplate thickness.

(2) A steel sleeve shall be provided to span across the openings to the opposite faceplates. The sleeve nominal yield strength and thickness shall match or exceed the faceplate nominal yield strength and thickness, respectively. The sleeve shall be connected to both faceplates using CJP groove welds.

(3) The steel sleeve shall be anchored into the surrounding concrete in accordance with the requirements of Section N9.1.3, where the width-to-thickness ratio is calculated using the sleeve thickness instead of the faceplate thickness.

(4) On each face, a reinforcing flange, made from the same material as the section faceplate and extending beyond the opening perimeter by a distance equal to the section thickness for the interior region and half the section thickness for the connection region, shall be provided in one of the following ways:

(i) In the form of a doubler plate, mounted outboard of the faceplate and with the same thickness as the faceplate, wherein the doubler plate shall be joined with the sleeve using a CJP groove weld around perimeter of the sleeve, and the doubler plate shall be joined with the faceplate using the maximum size fillet weld permitted by the *Specification* at its outer perimeter;

(ii) In the form of an independent reinforcing plate, with thickness equal to at least 1.25 times the surrounding faceplate, which shall be joined using a CJP groove weld with the sleeve at its inner perimeter and with the surrounding faceplate at its outer perimeter. An additional fillet weld, with leg size equal to the difference between the thicknesses of the independent reinforcing plate and the surrounding faceplate, shall be provided at the outer perimeter of the reinforcing plate if the thickness difference is equal to or greater than $^3/_{16}$ in. (5 mm).

7b. Design and Detailing Requirements for Large Openings

At the boundary of large openings, detailing is permitted to be provided to achieve either a free edge or a fully developed SC structural element. Design and detailing shall be as follows:

(a) Design and detailing with free edge at the perimeter of large openings

(1) The size of the opening modeled for analysis purposes shall be larger than the physical opening such that it extends to where the faceplates are fully developed away from the boundary of the opening.
(2) No reductions shall be applied to the available strengths of the panel sections in the vicinity of the as-modeled opening.
(3) Reentrant corners of noncircular or non-oval openings shall have corner radii not less than four times the faceplate thickness.
(4) The first row of ties around the opening shall be located at a distance from the opening no greater than one-quarter of the SC section thickness, t_{sc}.

(b) Design and detailing with fully developed edge at the perimeter of large openings

Fully developed SC structural elements around large openings shall be modeled and designed considering the physical boundary of the opening and shall follow the provisions for design and detailing with fully developed edge at the perimeter of small openings.

User Note: Small openings are not modeled in the analysis. However, the prescriptive detailing requirements of this section will provide SC panel sections with adequate strength and reduced local stress concentrations around small openings. Large openings have additional modeling requirements as discussed in Commentary Section N9.2.1 and should be detailed in accordance with Section N9.1.7b by taking into account the nature of boundary conditions provided around the opening.

During its placement, the fresh concrete can exert significant hydrostatic pressure on the sleeves for large openings. Accordingly, the sleeves should be evaluated for the associated non-uniform radial pressure loading.

7c. Design and Detailing Requirements for a Bank of Small Openings

It is permitted to neglect the effect of a bank of small openings if each of them individually meets the relevant exemption and detailing requirements in Section N9.1.7a, and if the center-to-center spacing between all such small openings exceeds the SC structural element thickness. The following detailing requirements shall be followed when these requirements are not satisfied.

(1) The region affected by a concentrated bank of small openings shall be treated as a large opening when the smallest clear distance between adjacent small openings is between t_{sc} and $2t_{sc}$ for the interior region and $0.5t_{sc}$ and $1.5t_{sc}$ for the connection region.
(2) The bank of small openings shall be reinforced using a single reinforcing plate that:
   (a) incorporates all sleeves within the bank of openings;
   (b) meets the requirements of Section N9.1.7a(b); and
   (c) extends the minimum required distance beyond the perimeter of the outermost openings.

If the longest and shortest dimensions of the bank of openings exceed $2t_{sc}$ and t_{sc}, respectively, then it shall be analyzed per Section N9.1.7b as an equivalent large opening that circumscribes the outermost sleeves.

N9.2. ANALYSIS REQUIREMENTS

1. General Provisions

The following provisions shall apply to the analysis of SC structural elements.

(a) SC structural elements shall be analyzed using elastic, three-dimensional, thick-shell, or solid finite elements.

> **User Note:** Guidance for finite element analysis or modeling, including the refined mesh around openings, are provided in the Commentary to this section. Section N9.1.7 provides modeling and detailing requirements for small openings and large openings.

(b) Second-order effects shall be addressed in accordance with Section N9.1.2b.
(c) Finite element analyses involving accident thermal conditions shall be conducted in accordance with Section N9.2.4.
(d) The viscous damping ratio for safe shutdown earthquake (SSE) level seismic analysis shall not exceed 5% for the determination of required strengths for SC structural elements.

2. Effective Stiffness for Analysis

(a) The effective flexural stiffness for the analysis of SC structural elements shall be determined as follows:

$$(EI)_{eff} = \left(E_s I_s + c_2 E_c I_c\right)\left(1 - \frac{\Delta T_{savg}}{150}\right) \ge E_s I_s \text{, kip-in.}^2\text{/ft} \qquad \text{(A-N9-8)}$$

$$(EI)_{eff} = \left(E_s I_s + c_2 E_c I_c\right)\left(1 - \frac{\Delta T_{savg}}{83}\right) \ge E_s I_s \text{, (N-mm}^2\text{/m)} \qquad \text{(A-N9-8M)}$$

where

E_c = modulus of elasticity of concrete
= $w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043 w_c^{1.5}\sqrt{f_c'}$, MPa)

I_c = moment of inertia of concrete infill per unit width
= $l\left(t_c^3/12\right)$, in.^{4}/ft (mm^4/m)

I_s = moment of inertia per unit width of faceplates (corresponding to the condition when the concrete is fully cracked)
= $l t_p (t_{sc} - t_p)^2/2$, in.^{4}/ft (mm^4/m)

c_2 = calibration constant for determining effective flexural stiffness
= $0.48\rho' + 0.10$

f_c' = specified compressive strength of concrete, ksi (MPa)

l = 12 in./ft (1000 mm/m)

n = modular ratio of steel and concrete
= E_s/E_c

t_c = concrete infill thickness, in. (mm)

t_{sc} = SC section thickness, in. (mm)

ρ = reinforcement ratio
= $2t_p/t_{sc}$

ρ' = stiffness-adjusted reinforcement ratio
= ρn

ΔT_{savg} = average of the maximum surface temperature increases for the faceplates due to accident thermal conditions, °F (°C)

User Note: Equation A-N9-8 (A-N9-8M) is based on the stiffness of the cracked transformed section, including contributions of the faceplates and the cracked concrete infill. It also includes the reduction in flexural stiffness due to additional concrete cracking resulting from thermal accident conditions. For operating thermal conditions, it is reasonable to assume no further reduction due to thermal effects, i.e., $\Delta T_{savg} = 0$, because the gradients are small and they develop over significant time.

(b) The effective in-plane shear stiffness per unit width, $(GA)_{eff}$, for all load combinations that do not involve accident thermal loading shall be based on the required membrane in-plane shear strength per unit width, S_{rxy}, in the panel sections.

(1) If $S_{rxy} \le S_{cr}$

$$(GA)_{eff} = (GA)_{uncr}$$
$$= G_s A_s + G_c A_c \qquad \text{(A-N9-9)}$$

where

A_c = area of concrete infill per unit width
= lt_c, in.^{2}/ft (mm^2/m)

A_s = gross area of faceplates per unit width
= $l(2t_p)$, in.^{2}/ft (mm^2/m)

G_c = shear modulus of concrete
= $772\sqrt{f_c'}$, ksi ($2000\sqrt{f_c'}$, MPa)

G_s = shear modulus of elasticity of steel
= 11,200 ksi (77 200 MPa) for carbon steel and duplex stainless steel
= 10,800 ksi (74 500 MPa) for austenitic stainless steel

$(GA)_{uncr}$ = in-plane shear stiffness per unit width of uncracked composite SC panel section, kip/ft (N/m)
= $G_s A_s + G_c A_c$

S_{rxy} = required membrane in-plane shear strength per unit width in the panel section, kip/ft (N/m)

S_{cr} = in-plane shear force per unit width at concrete cracking threshold, kip/ft (N/m)

$$= \frac{0.063\sqrt{f_c'}}{G_c}(GA)_{uncr} \qquad \text{(A-N9-10)}$$

$$= \frac{0.17\sqrt{f_c'}}{G_c}(GA)_{uncr} \qquad \text{(A-N9-10M)}$$

f_c' = specified compressive strength of concrete, ksi (MPa)

l = 12 in./ft (1000 mm/m)

(2) If $S_{cr} < S_{rxy} \le 2S_{cr}$

$$(GA)_{eff} = (GA)_{uncr} - \left(\frac{(GA)_{uncr} - (GA)_{cr}}{S_{cr}}\right)(S_{rxy} - S_{cr}) \qquad \text{(A-N9-11)}$$

where

$$(GA)_{cr} = 0.5\bar{\rho}^{-0.42} G_s A_s \qquad \text{(A-N9-12)}$$

$\bar{\rho}$ = strength-adjusted reinforcement ratio

$$= \frac{A_s F_y}{31.6 A_c \sqrt{f_c'}} \qquad \text{(A-N9-13)}$$

$$= \frac{A_s F_y}{83 A_c \sqrt{f_c'}} \quad \text{(A-N9-13M)}$$

(3) If $S_{rxy} > 2S_{cr}$

$$(GA)_{eff} = (GA)_{cr} \quad \text{(A-N9-14)}$$

(c) The effective in-plane shear stiffness per unit width, $(GA)_{eff}$, for all load combinations involving accident thermal conditions shall account for the effects of concrete cracking by setting $(GA)_{eff}$ equal to $(GA)_{cr}$ determined using Equation A-N9-12.

(d) SC structural element connections shall be classified as rigid or pinned for out-of-plane moment transfer in accordance with Section N9.4.1 and modeled as per the classification.

3. Geometric and Material Properties for Finite Element Analysis

Geometric and material properties of the SC structural elements shall be modeled in the elastic finite element analyses as follows:

(a) The as-modeled poisson's ratio, ν_m, thermal expansion coefficient, α_m, and thermal conductivity, k_m, used in the elastic finite element analysis of SC panel sections shall be taken as that of the concrete.

(b) The as-modeled thickness of a SC panel section, t_m, and the material elastic modulus used in elastic finite element analysis of SC panel sections, E_m, shall be established through calibration to match the effective stiffness values for analysis, $(EI)_{eff}$ and $(GA)_{eff}$, defined in Section N9.2.2.

(c) The as-modeled material density used in elastic finite element analysis of the SC panel sections, γ_m, shall be established through calibration after the model section thickness, t_m, has been matched to the mass of the SC section.

(d) The as-modeled specific heat used in elastic finite element analysis of SC panel sections, c_m, shall be established through calibration after establishing density such that the model specific heat equals the specific heat of the concrete infill.

4. Analyses Involving Normal Operating and Accident Thermal Conditions

4a. Requirements for Normal Operating Thermal Conditions

For normal operation or other long-term period exposure:

(a) The steel surface temperatures shall not exceed 180°F (82°C) except for local areas such as around penetrations, which are permitted to have increased temperatures not to exceed 230°F (110°C); and

(b) The maximum strain in faceplates shall not exceed ε_y under normal thermal gradients.

4b. Requirements for Accident Thermal Conditions

For accident or any other short-term period exposure, the steel surface temperatures shall not exceed 570°F (300°C). Local areas are permitted to reach steel surface temperatures up to 800°F (430°C) from steam or water jets in the event of pipe failure.

Higher steel surface temperatures than those provided in this section are permitted if reduction in strength determined by testing or other rational criteria is applied to design. In addition, the engineer of record shall justify, by testing or other rational criteria, that increased temperatures do not cause deterioration of SC structural elements with or without the postulated loads.

Analyses for load combinations involving accident thermal conditions shall include heat transfer analyses. The heat transfer analysis results shall be used to define thermal loading for the structural analyses.

Heat transfer analyses shall be conducted using the geometric and material properties specified in Section N9.2.3 to estimate the temperature histories and through-section temperature profiles produced by the thermal accident conditions. These temperature histories and through section temperature profiles shall be considered in the structural finite element analyses.

The required out-of-plane flexural strengths per unit width, M_{rx} and M_{ry}, in the SC structural element interior regions caused by the thermal gradients shall not exceed $M_{r\text{-}th}$, where

$$M_{r\text{-}th} = (EI)_{eff}\left(\frac{\alpha_s \Delta T_{sg}}{t_{sc}}\right) \qquad \text{(A-N9-15)}$$

where

$(EI)_{eff}$ = effective flexural stiffness for analysis of SC structural elements per unit width, kip-in.^{2}/ft (N-mm^2/m)

α_s = thermal expansion coefficient of faceplate in ºF^{-1} (ºC^{-1})

ΔT_{sg} = maximum temperature difference between faceplates due to accident thermal conditions in ºF (ºC)

User Note: The $M_{r\text{-}th}$ value in Equation A-N9-15 considers full flexural restraint and accounts for the relief from concrete cracking that limits the thermally induced moments. The analysis results for thermal loads may predict moments higher than $M_{r\text{-}th}$ defined above if (a)it does not directly account for the self-limiting effect due to concrete cracking, and/or (b) ΔT_{sg} is very large such that α_s times ΔT_{sg} exceeds the material yield strain. For the connection regions, the out-of-plane moment demands are determined by the finite element analyses, and the upper limit from Equation A-N9-15 does not apply.

5. Determination of Required Strengths

In-plane membrane forces, out-of-plane moments, and out-of-plane shear forces shall be determined by an elastic finite element analysis.

The required strength for each load effect shall be calculated by averaging the load effect over panel sections that are no larger than twice the section thickness in length and width. In the vicinity of openings and penetrations, and in connection regions, the required strength shall be calculated by averaging the load effect over panel sections no larger than the section thickness in length and width.

The required strengths for the panel sections of SC structural elements for each load effect shall be denoted as follows:

M_{rx} = required out-of-plane flexural strength per unit width in direction x, kip-in./ft (N-mm/m)
M_{ry} = required out-of-plane flexural strength per unit width in direction y, kip-in./ft (N-mm/m)
M_{rxy} = required twisting moment strength per unit width, kip-in./ft (N-mm/m)
S_{rx} = required membrane axial strength per unit width in direction x, kip/ft (N/m)
S_{ry} = required membrane axial strength per unit width in direction y, kip/ft (N/m)
S_{rxy} = required membrane in-plane shear strength per unit width, kip/ft (N/m)
V_{rx} = required out-of-plane shear strength per unit width along edge parallel to direction x, kip/ft (N/m)
V_{ry} = required out-of-plane shear strength per unit width along edge parallel to direction y, kip/ft (N/m)
x, y = local coordinate axes in the plane of the panel section associated with the finite element model

N9.3. DESIGN OF SC STRUCTURAL ELEMENTS

The tensile strength contribution of concrete infill and the contribution of steel ribs to the available strengths of SC structural elements shall be neglected.

1. Uniaxial Tensile Strength

The available uniaxial tensile strength per unit width of SC structural element panel sections shall be determined in accordance with *Specification* Chapter D. Where holes are present in faceplates, the available rupture strength shall be greater than the available yield strength.

2. Compressive Strength

The available compressive strength per unit width of SC structural element panel sections shall be determined in accordance with *Specification* Section I2.1b with the faceplates taking the place of the steel shape.

The terms listed below are used in addition to or as replacements of those used in the Specification Section I2.1b:

P_{no} = nominal compressive strength per unit width, kip/ft (N/m)

$$= F_y A_{sn} + 0.85 f_c' A_c \quad \text{(A-N9-16)}$$

P_e = elastic critical buckling load per unit width, kip/ft (N/m)

$$= \pi^2 (EI)_{eff} / L_c^2 \quad \text{(A-N9-17)}$$

A_c = area of the concrete infill per unit width, in.^{2}/ft (mm^2/m)

= lt_c, in.^{2}/ft (mm^2/m)

A_{sn} = net area of faceplates per unit width, in.^{2}/ft (mm^2/m)

E_c = modulus of elasticity of concrete

= $w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043 w_c^{1.5}\sqrt{f_c'}$, MPa)

$(EI)_{eff}$ = effective SC stiffness per unit width for buckling evaluation, kip-in.^{2}/ft (N-mm^2/m)

$$= E_s I_s + 0.60\, E_c I_c \quad \text{(A-N9-18)}$$

I_c = moment of inertia of concrete infill per unit width

= $lt_c^3/12$, in.^{4}/ft (mm^4/m)

I_s = moment of inertia per unit width of faceplates (corresponding to the condition when concrete is fully cracked)

= $l\left[t_p (t_{sc} - t_p)^2 / 2\right]$, in.^{4}/ft (mm^4/m)

L_c = effective length of member, in. (mm)

f_c' = specified compressive strength of concrete, ksi (MPa)

l = 12 in./ft (1000 mm/m)

3. Out-of-Plane Flexural Strength

The design flexural strength, $\phi_b M_n$, and the allowable flexural strength, M_n/Ω_b, per unit width of SC structural element panel sections shall be determined for the limit state of yielding as follows:

$$M_n = F_y (A_s^F)(t_{sc}) \quad \text{(A-N9-19)}$$

$\phi_b = 0.90$ (LRFD) $\qquad \Omega_b = 1.67$ (ASD)

where

A_s^F = gross area of faceplate in tension due to flexure per unit width, in.^{2}/ft (mm^2/m)
F_y = specified minimum yield stress of faceplate, ksi (MPa)
t_{sc} = SC section thickness, in. (mm)

4. In-Plane Shear Strength

The design in-plane strength per unit width, $\phi_{vi}V_{ni}$, and the allowable in-plane shear strength per unit width, V_{ni}/Ω_{vi}, of panel sections shall be determined for the limit state of yielding of the faceplates as follows:

$$V_{ni} = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}} A_s F_y \le A_s F_y \qquad \text{(A-N9-20)}$$

ϕ_{vi} = 0.90 (LRFD) Ω_{vi} = 1.67 (ASD)

where
A_s = gross area of faceplates per unit width
= $l(2t_p)$, in.^{2}/ft (mm^2/m)
F_y = specified minimum yield stress of faceplates, ksi (MPa)
V_{ni} = nominal in-plane shear strength per unit width of SC panel section, kip/ft (N/m)
l = 12 in./ft (1000 mm/m)
$K_s = G_s A_s$
$K_{sc} = \dfrac{0.7(E_c A_c)(E_s A_s)}{4E_s A_s + E_c A_c}$

5. Out-of-Plane Shear Strength

The nominal out-of-plane shear strength per unit width shall be established by one of the following:

(1) Project specific large-scale out-of-plane shear tests
(2) Test results
(3) The provisions of this section

The design out-of-plane shear strength per unit width, $\phi_{vo}V_{no}$, and the allowable out-of-plane shear strength per unit width, V_{no}/Ω_{vo}, of panel sections shall be determined as follows:

ϕ_{vo} (LRFD) = 0.90 for SC panel sections with yielding ties, except when Section N9.3.5(b) applies and V_{conc} exceeds V_s

= 0.75 for all other cases

Ω_{vo} (ASD) = 1.67 for SC panel sections with yielding ties, except when

Section 9.3.5(b) applies and V_{conc} exceeds V_s

=2.00 for all other cases

User Note: The classification of out-of-plane shear reinforcement (in the form of ties—namely, structural steel shapes, frames, or tie bars embedded in the concrete infill) as yielding shear reinforcement or nonyielding shear reinforcement should be done in accordance with Section N9.1.5a.

(a) The nominal out-of-plane shear strength per unit width of SC panel sections, V_{no}, with shear reinforcement spacing no greater than half of the section thickness shall be calculated as follows:

$$V_{no} = V_{conc} + V_s \tag{A-N9-21}$$

where

V_{conc} = nominal out-of-plane shear strength contributed by concrete per unit width of SC panel section, kip/ft (N/m)

$= 0.063(f'_c)^{0.5} t_c l$ (A-N9-22)

$= 0.166(f'_c)^{0.5} t_c l$ (A-N9-22M)

V_s = nominal out-of-plane shear strength contributed by steel per unit width of SC panel section, kip/ft (N/m)

$\xi\left(\frac{t_c}{s_{tl}}\right)F_t\left(\frac{l}{s_{tt}}\right)$ (A-N9-23)

F_t = nominal tensile strength of tie, kips (N)

l = 12 in./ft (1000 mm/m)

s_{tl} = spacing of shear reinforcement along the direction of one-way shear, in. (mm)

s_{tt} = spacing of shear reinforcement transverse to the direction of one-way shear, in. (mm)

t_c = concrete infill thickness, in. (mm) = $t_{sc} - 2t_p$, in. (mm)

ξ = 1.0 for yielding shear reinforcement

= 0.5 for nonyielding shear reinforcement

User Note: The "nominal tensile strength" value is equal to: (1) F_yA_g for yielding shear reinforcement (i.e., when the tensile yielding limit state controls), and (2) F_{nr} for non-yielding shear reinforcement (i.e., when the tie rupture strength or its connection strength to the faceplate controls).

(b) The nominal out-of-plane shear strength per unit width of SC panels, V_{no}, with shear reinforcement spaced greater than half the section thickness shall be the greater of V_{conc} and V_s. V_{conc} shall be calculated using Equation A-N9-22 or Equation A-N9-22M, and V_s shall be calculated using Equation A-N9-23, taking both ξ and (t_c/s_{tl}) as 1.0.

6. Interaction Criteria for SC Structural Elements Subjected to Concurrent In-Plane and Out-of-Plane Forces

User Note: This section provides interaction equations for verifying the adequacy of SC structural elements subjected to concurrent forces due to individual load cases and specified load combinations. It is noted that the interaction equations are valid for load combinations involving both operating thermal and accident thermal load cases.

6a. Interfacial Shear and Out-of-Plane Shear Forces

The interaction of out-of-plane shear forces shall be limited by the following:

(a) If the required out-of-plane shear strength per unit width for both the x and y axes, V_{rx} and V_{ry}, is greater than the available out-of-plane shear strength contributed by the concrete per unit width of SC panel section, $V_{c\ conc}$, and the out-of-plane shear reinforcement is spaced no greater than half the section thickness:

For nonyielding shear reinforcement:

$$\left[\left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_x + \left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_y\right]^{5/3} + \left[\frac{\sqrt{V_{rx}^2 + V_{ry}^2}\Big/(0.9t_{sc})}{\Psi\left(lQ_{cv}^{avg}/s^2\right)}\right]^{5/3} \le 1.0 \qquad \text{(A-N9-24a)}$$

For yielding shear reinforcement:

$$\left[\left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_x + \left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_y\right]^{2} + \left[\frac{\sqrt{V_{rx}^2 + V_{ry}^2}\Big/(0.9t_{sc})}{\Psi\left(lQ_{cv}^{avg}/s^2\right)}\right]^{2} \le 1.0 \qquad \text{(A-N9-24b)}$$

where

Q_{cv}^{avg} = weighted average of the available interfacial shear strengths of a group of shear connectors that accounts for tributary areas of each type of connector, kips (N)

V_c = available out-of-plane shear strengths per unit width of SC panel section in local x (V_{cx}) and y (V_{cy}) directions, kip/ft (N/m)

$V_{c\ conc}$ = available out-of-plane shear strength contributed by concrete per unit width of SC panel section, kip/ft (N/m)

V_r = required out-of-plane shear strength per unit width of SC panel section in local x (V_{rx}) and y (V_{ry}) directions using LRFD or ASD load combinations, kip/ft (N/m)

l = 12 in./ft (1000 mm/m)

s = spacing of shear connectors, in. (mm)
t_{sc} = SC section thickness, in. (mm)
x = subscript relating symbol to the local x-axis
y = subscript relating symbol to the local y-axis
Ψ = 1.0 for panel sections with yielding shear connectors
= 0.5 for panel sections with nonyielding shear connectors

For design according to *Specification* Section B3.1 (LRFD)

V_c = $\phi_{vo}V_{no}$, kip/ft (N/m), where V_{no} is calculated in accordance with Section N9.3.5
$V_{c\ conc}$ = $\phi_{vo}V_{conc}$, kip/ft (N/m), where V_{conc} is calculated in accordance with Section N9.3.5
V_r = required out-of-plane shear strength per unit width of SC panel section in local x (V_{rx}) and y (V_{ry}) directions using LRFD load combinations, kip/ft (N/m)
ϕ_{vo} = 0.75

For design according to *Specification* Section B3.2 (ASD)

V_c = V_{no}/Ω_{vo}, kip/ft (N/m), where V_{no} is calculated in accordance with Section N9.3.5
$V_{c\ conc}$ = $V_{conc}/\ \Omega_{vo}$, kip/ft (N/m), where V_{conc} is calculated in accordance with Section N9.3.5
V_r = required out-of-plane shear strength per unit width of SC panel section in local x (V_{rx}) and y (V_{ry}) directions using ASD load combinations, kip/ft (N/m)
Ω_{vo} = 2.00

(b) If the available strength, V_c, is governed by the steel contribution alone and the out-of-plane shear reinforcement is spaced greater than half the section thickness, $V_{c\ conc}$ shall be taken as zero in Equations A-N9-24a and A-N9-24b.

6b. In-Plane Membrane Forces and Out-of-Plane Moments

The design adequacy of the panel sections subjected to the three in-plane required membrane strengths (S_{rx}, S_{ry}, S_{rxy}) and three out-of-plane required flexural or twisting strengths (M_{rx}, M_{ry}, M_{rxy}) shall be evaluated for each notional half of the SC section that consists of one faceplate and half the concrete thickness.

For each notional half, the interaction shall be limited by Equations A-N9-25 to A-N9-27. These equations shall be used with the maximum and minimum required principal in-plane strengths per unit width for the notional half of the SC panel section, $S_{r,max}$ and $S_{r,min}$, calculated using Equations A-N9-28 to A-N9-31.

(a) For $S_{r,max} + S_{r,min} \geq 0$

$$\alpha\left(\frac{S_{r,max} + S_{r,min}}{2V_{ci}}\right) + \left(\frac{S_{r,max} - S_{r,min}}{2V_{ci}}\right) \leq 1.0 \qquad \text{(A-N9-25)}$$

(b) For $S_{r,max} > 0$ and $S_{r,max} + S_{r,min} < 0$

$$\frac{S_{r,max}}{V_{ci}} - \beta\left(\frac{S_{r,max} + S_{r,min}}{V_{ci}}\right) \leq 1.0 \qquad \text{(A-N9-26)}$$

(c) For $S_{r,max} \leq 0$ and $S_{r,min} \leq 0$

$$-\beta\left(\frac{S_{r,min}}{V_{ci}}\right) \leq 1.0 \qquad \text{(A-N9-27)}$$

where

α = V_{ci}/T_{ci}

β = V_{ci}/P_{ci}

$S_{r,max}$, $S_{r,min}$ =

$$\frac{S'_{rx} + S'_{ry}}{2} \pm \sqrt{\left(\frac{S'_{rx} - S'_{ry}}{2}\right)^2 + \left(S'_{rxy}\right)^2} \qquad \text{(A-N9-28)}$$

$$S'_{rx} = \frac{S_{rx}}{2} \pm \frac{M_{rx}}{j_x t_{sc}} \qquad \text{(A-N9-29)}$$

$$S'_{ry} = \frac{S_{ry}}{2} \pm \frac{M_{ry}}{j_y t_{sc}} \qquad \text{(A-N9-30)}$$

$$S'_{rxy} = \frac{S_{rxy}}{2} \pm \frac{M_{rxy}}{j_{xy} t_{sc}} \qquad \text{(A-N9-31)}$$

S'_{rx} = required membrane axial strength per unit width in direction x for each notional half of SC panel section, kip/ft (N/m)

S'_{ry} = required membrane axial strength per unit width in direction y for each notional half of SC panel section, kip/ft (N/m)

S'_{rxy} = required membrane in-plane shear strength per unit width for each notional half of SC panel section, kip/ft (N/m)

j_x = parameter for distributing required flexural strength, M_{rx}, into the corresponding membrane force couples acting on each notional half of SC panel section

= 0.9 if $S_{rx} > -0.6P_{no}$

= 0.67 if $S_{rx} \leq -0.6P_{no}$

j_y = parameter for distributing required flexural strength, M_{ry}, into the corresponding membrane force couples acting on each notional half of SC panel section

= 0.9 if $S_{ry} > -0.6P_{no}$

$= 0.67$ if $S_{ry} \leq -0.6P_{no}$

j_{xy} = parameter for distributing required flexural strength, M_{rxy}, into the corresponding membrane force couples acting on each notional half of SC panel section
$= 0.67$

P_{no} = nominal compressive strength per unit width calculated using Equation A-N9-16, kip/ft (N/m)

Alternatively, for each notional half, the interaction shall be limited directly with the required in-plane membrane strengths per unit width (S'_{rx}, S'_{ry}, S'_{rxy}), using Equations A-N9-32 to A-N9-34. S'_{rx}, S'_{ry} and S'_{rxy} shall be calculated using Equations A-N9-29 to A-N9-31.

(a) For $S'_{rx} + S'_{ry} \geq 0$

$$\left(1-\alpha^2\right)\left(\frac{S'_{rx}+S'_{ry}}{2V_{ci}}\right)^2 + \alpha\left(\frac{S'_{rx}+S'_{ry}}{V_{ci}}\right) + \left[\frac{\left(S'_{rxy}\right)^2 - S'_{rx}S'_{ry}}{V_{ci}^2}\right] \leq 1.0 \qquad \text{(A-N9-32)}$$

(b) For $0 \geq S'_{rx} + S'_{ry} \geq -P_{ci}$

$$\beta\left(1-\beta\right)\left(\frac{S'_{rx}+S'_{ry}}{V_{ci}}\right)^2 + \left(1-2\beta\right)\left(\frac{S'_{rx}+S'_{ry}}{V_{ci}}\right) + \left[\frac{\left(S'_{rxy}\right)^2 - S'_{rx}S'_{ry}}{V_{ci}^2}\right] \leq 1.0 \qquad \text{(A-N9-33)}$$

(c) For $-P_{ci} \geq S'_{rx} + S'_{ry}$

$$\beta^2\left[\frac{\left(S'_{rxy}\right)^2 - S'_{rx}S'_{ry}}{V_{ci}^2}\right] - \beta\left(\frac{S'_{rx}+S'_{ry}}{V_{ci}}\right) \leq 1.0 \qquad \text{(A-N9-34)}$$

where

P_{ci} = available compressive strength per unit width for each notional half of SC panel section, kip/ft (N/m)

T_{ci} = available tensile strength per unit width for each notional half of SC panel section, kip/ft (N/m)

V_{ci} = available in-plane shear strength per unit width for each notional half of SC panel section, kip/ft (N/m)

For design according to *Specification* Section B3.1 (LRFD)

P_{ci} = $\phi_{ci}P_{no}/2$, kip/ft (N/m), where P_{no} is calculated using the nominal section compressive strength in accordance with Section N9.3.2

T_{ci} = $\phi_{ti}T_{ni}/2$, kip/ft (N/m)

T_{ni} = nominal tensile strength per unit width of SC panel section determined in accordance with Section N9.3.1, kip/ft (N/m)

V_{ci} = $\phi_{vs}V_{ni}/2$, kip/ft (N/m), where V_{ni} is calculated using the nominal in-plane shear strength in accordance with Section N9.3.4

ϕ_{ci} = 0.80
ϕ_{ti} = 1.00
ϕ_{vs} = 0.95

For design according to *Specification* Section B3.2 (ASD)

P_{ci} = $P_{no}/(2\Omega_{ci})$, kip/ft (N/m), where P_{no} is calculated using the nominal section compressive strength in accordance with Section N9.3.2
T_{ci} = $T_{ni}/(2\Omega_{ti})$, kip/ft (N/m)
T_{ni} = nominal tensile strength per unit width of SC panel section determined in accordance with Section N9.3.1, kip/ft (N/m)
V_{ci} = $V_{ni}/(2\Omega_{vs})$, kip/ft (N/m), where V_{ni} is calculated using the nominal in-plane shear strength in accordance with Section N9.3.4
Ω_{ci} = 1.88
Ω_{ti} = 1.50
Ω_{vs} = 1.58

User Note: Use of the alternative interaction equations, A-N9-32, A-N9-33, and A-N9-34, may result in total interaction values that are negative; such instances should be interpreted as the element having satisfied the applicable interaction equation.

7. Strength of Composite Members in Combination with SC Structural Elements

Composite members are permitted to be used in conjunction with SC structural elements. They shall be designed in accordance with Chapter NI.

N9.4. DESIGN OF SC STRUCTURAL ELEMENT CONNECTIONS

This section addresses design requirements for connections involving SC structural elements, either with other SC structural elements or with reinforced concrete (RC) structural elements.

User Note: Examples of such connections include the following:

(a) Co-planar splices between SC walls, SC slabs, or SC basemat sections;
(b) Co-planar splices between SC wall, SC slab, or SC basematand corresponding reinforced concrete (RC) elements;
(c) Connections at the intersections of SC walls and SC slabs, and SC walls and SC basemats;
(d) Connections at the intersection of SC and RC walls;
(e) Anchorage of SC walls to RC basemats; and
(f) Connections between SC walls and RC slabs.

1. General Provisions

Splice connections shall be rigid for out-of-plane moment transfer. Wall-to-slab connections shall be rigid or pinned, consistent with the analysis model used.

Connectors shall consist of steel headed stud anchors, anchor rods, tie bars, reinforcing bars and dowels, post-tensioning bars, shear lugs, embedded steel shapes, welds and bolts, reinforcing steel mechanical couplers, and direct bearing in compression. Force transfer mechanisms involving connectors of the same type shall be provided for each type of connection interface force. Direct bond transfer between the faceplate and concrete shall not be considered as a valid connector or force transfer mechanism.

User Note: If more than one force transfer mechanism is possible, the one that provides the greatest strength is assumed to be the governing force transfer mechanism. For additional details and SC wall/slab connection design examples, refer to AISC Design Guide 32, *Design of Modular Steel-Plate Composite Walls for Safety-Related Nuclear Facilities*.

1a. Required Strength

The required strength for the connections shall be determined as:

(a) 125% of the smaller of the corresponding nominal strengths of the connected parts, or

(b) 200% of the required strength due to seismic loads plus 100% of the required strength due to nonseismic loads (including thermal loads).

User Note: Connections designed for required strength as per option (a) develop the expected available strength of the weaker of the connected parts. Connections designed for required strength as per option (b) develop overstrength with respect to the connection design demands, while ensuring that ductile limit states govern the connection strength. Option (a) is preferred. Where option (a) is not practical, option (b) may be used.

1b. Available Strength

The available strength shall be calculated using the applicable force transfer mechanism and the available strength of the connectors contributing to the force transfer mechanism. The available strength for connectors shall be determined as follows:

(a) For steel headed stud anchors, the available strength shall be determined in accordance with *Specification* Section I8.3 with modifications in Chapter NI.

(b) For welds and bolts, the available strength shall be determined in accordance with Chapter NJ.

(c) For compression transfer via direct bearing on concrete, the available strength is determined in accordance with *Specification* Section I6.3a.

(d) For shear friction load transfer mechanism, the available strength is determined in accordance with ACI 349 or ACI 349M, Section 11.7.

(e) For embedded shear lugs and shapes, the available strength is determined in accordance with ACI 349 or ACI 349M, Appendix D.

(f) For anchor rods, the available strength is determined from ACI 349 or ACI 349M, Appendix D.

(g) For lap splices of reinforcing bars with faceplates, the available strength is determined as the yield strength of lapped reinforcing bars provided that the requirements of Section N9.4.2 are satisfied.

2. Lap Splicing of Reinforcing Bars with Faceplates

Lap splicing of reinforcing bars with SC section faceplates shall meet the following requirements:

(a) Dowels larger than 1.4 in. (36 mm) diameter are not permitted for splices.

(b) The embedment length of the dowels within the SC structural element shall be at least the lap splice length calculated per ACI 349 or ACI 349M.

(c) If steel headed stud anchors are used, the dowels shall be located within the length of, and confined by, the steel headed stud anchors. The minimum spacing between the dowels to the closest faceplate shall be the dowel bar diameter.

(d) The available interfacial shear strength of the steel headed stud anchors along the dowel embedment length shall be greater than or equal to 125% of the nominal yield strength of the reinforcing steel.

Add the following Appendix.

APPENDIX N10

SPECIAL PROVISIONS FOR IMPACTIVE AND IMPULSIVE LOADS

This appendix addresses the design, analysis, and detailing requirements for impactive and impulsive loads for structural steel elements, composite members (for example, composite beams and columns), and SC structural elements. The provisions of this appendix apply to those structural elements directly affected by the impactive and impulsive loads. Because of their differing behavior characteristics, separate sections are provided in this appendix for structural steel, composite members, and steel plate, and for SC structural elements.

User Note: Examples of impactive loads include tornado-generated missiles, whipping pipes, aircraft missiles (this can be either design-basis or beyond design-basis load), fuel cask drop and other internal and external missiles.

Examples of impulsive loads include jet impingement, blast pressure, compartment pressurization and pipe-whip restraint reactions (in terms of how such reactions affect the structure that supports the impacted structural element).

The appendix is organized as follows:

N10.1. General Provisions
N10.2. Analysis, Design, and Detailing of Structural Steel, Composite Members, and Steel Plate
N10.3. Analysis, Design, and Detailing of SC Structural Elements

N10.1. GENERAL PROVISIONS

1. Additional Material Requirements

Additional material requirements for structural elements subjected to impactive and impulsive loadings shall be as follows:

(a) The specification of the material of those structures or structural elements that are subjected to impactive and/or impulsive loads shall comply with Section NA3.1. Welds subject to impactive and/or impulsive loads shall comply with Section NJ2.6.
(b) Bolts and threaded parts shall be in accordance with Section NJ3.14.
(c) The structural documents and specifications shall meet Section NA4.

2. Dynamic Strength Increase

It is permitted to consider strain rate-adjusted material strengths for structural steel, reinforcing steel, and concrete materials. The material strength increase shall be based on applicable experimental data. The Dynamic Increase Factors (DIF) specified in Table A-N10.1.1 are permitted for use in the absence of experimental data.

In case of elastic response, the DIF value shall be limited to 1.0 for all materials if the calculated dynamic load factor for the impactive or impulsive loading is less than 1.2.

TABLE A-N10.1.1
Dynamic Increase Factors (DIF)

Material	DIF	
	Yield Strength	Ultimate Strength
Structural steel shapes	1.10	1.05
Carbon steel plate	1.20	1.10
Stainless steel plate	1.10	1.05
Reinforcing steel Grade 60 (420 MPa) Grade 80 (550 MPa)	 1.10 1.10	 1.05 1.05
Concrete compressive strength	NA	1.25
Concrete shear strength	NA	1.10

NA = not applicable

User Note: The DIF values in Table A-N10.1.1 are conservatively adopted from NEI 07-13, *Methodology for Performing Aircraft Impact Assessments for New Plant Designs*, Revision 8P.

3. Load Effects and Load Combinations

For each applicable load combination, the required strength or ductility of the affected structural elements for the impactive and impulsive loads shall be determined by considering all other applicable concurrently acting loads.

N10.2. ANALYSIS, DESIGN, AND DETAILING OF STRUCTURAL STEEL, COMPOSITE MEMBERS, AND STEEL PLATE

1. Compactness Requirements

For structural steel elements and composite members subject to flexure or compression due to impactive and impulsive load, the limiting width-to-thickness ratios of their compression elements shall not exceed the limiting values, λ_c, provided in Table A-N10.2.1. The R_y values necessary for determination of λ_c in Table A-N10.2.1 shall be obtained from Table A3.2 in the *Seismic Provisions*.

Structural elements in flexure only, or combined flexure and compression, shall conform to the lateral bracing requirements of *Specification* Appendix 1, Section 1.3.2c.

2. Local Response Evaluation

For impactive and impulsive targets consisting of steel plate, the required minimum thickness to prevent perforation under impactive loads shall be checked using project-specific test data or published formulas developed from validated test data.

Local response evaluation of composite members subjected to impactive loads shall be based on project-specific test data or published formulas developed from validated test data.

User Note: For nuclear safety-related applications, local response evaluation for impulsive loads is not required because the characteristics of the applicable impulsive loads are such that they cannot cause perforation.

3. Special Design and Detailing Requirements for Ductility

Design of structural steel elements and composite members for impactive and impulsive loads shall follow the material requirements of *Seismic Provisions* Section A3, and the general member and connection requirements of *Seismic Provisions* Sections D1 and D2 for highly ductile members, respectively.

4. Analysis Requirements for Verification of Structural Element Ductility

It is permitted to determine the load effects for impactive or impulsive forces using inelastic analysis. Design adequacy of structural elements subjected to these load effects shall be assessed by using one of the following three methods:

(a) If the target response remains elastic, the dynamic load effects of the impulsive or impactive loads shall be calculated using the applicable dynamic load factor (DLF). The calculated maximum elastic required strengths using this method shall not exceed the available strengths defined in Chapters ND to NJ.

(b) If the target response is in the inelastic range, use of a simplified single-degree-of-freedom analysis of the target, using either a bilinear or multi-linear resistance function, is permitted. The calculated maximum ductility ratio using this method, defined below as μ_r, shall not exceed the applicable permissible ductility ratio, μ_p, provided in Table A-N10.2.2.

The required ductility ratio, μ_r, shall be calculated as follows:

$$\mu_r = \frac{D_m}{D_y} \qquad \text{(A-N10-1)}$$

where

D_m = maximum deflection from analysis, in. (mm)

D_y = effective yield deflection, in. (mm)

The acceptance criteria for composite members shall be based on project-specific test data or applicable published analytical methods developed from validated test data.

(c) Alternatively, if the target response is in the inelastic range, use of a detailed nonlinear and inelastic finite element analysis is permitted for direct determination of maximum strains. The calculated maximum plastic tensile strain using this method shall not exceed 0.03 in./in. (mm/mm).

User Note: Analysis and design of structural elements subjected to impactive or impulsive loads requires subject matter expertise. In particular, implementation of option (c) is more involved because it requires accurate determination of the structural element's stress-strain curve and its maximum response. Peer review by independent subject matter expert(s) is recommended if option (c) is implemented.

The method per option (b) is easier to implement because it involves a simplified bilinear (or multilinear) resistance function of the structural element's load-displacement behavior that is based on an equivalent single-degree-of-freedom model (accordingly, the permissible ductility ratios in Table A-N10.2.2 have been conservatively specified). This method is based on similar provisions in UFC 3-340-03 (DOD, 2008), which requires determination of the structural element's resistance function by using its nominal yield strength times the dynamic increase factor and applicable strain-hardening effect. As defined and illustrated in UFC 3-340-02 (DOD, 2008), the effective yield point is taken as the intersection point of the line representing the initial equivalent stiffness with the horizontal line representing the plastic behavior (see commentary for further discussion). The associated effective yield displacement is used for implementation of option (b).

For all methods, the associated connections shall be designed such that their available strengths including the dynamic increase factor are greater than R_y times the nominal strength for LRFD and R_y/1.5 times the nominal strength for ASD of the connected structural element, where the R_y value corresponds to the material used in the connected structural element and is obtained from *Seismic Provisions* Table A3.2.

TABLE A-N10.2.1
Limiting Width-to-Thickness Ratios for Structural Steel and Composite Members

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio	Examples
				λ_c	

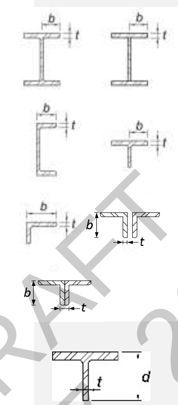
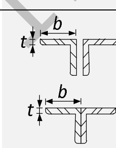
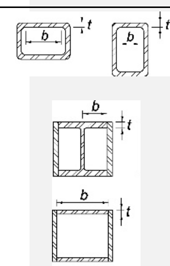
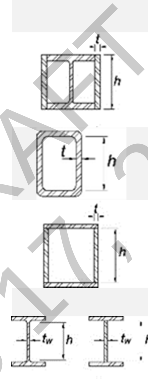
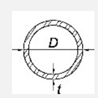
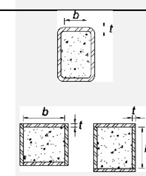
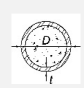
Unstiffened Elements	1	(1) Flanges of rolled or built-up I-shaped sections (2) Flange and stem of rolled or built-up tees (3) Flanges of rolled or built-up channels (4) Legs of single angles or double-angle members with separators (5) Outstanding legs of pairs of angles in continuous contact	b/t d/t	$0.30\sqrt{\frac{E}{R_y F_y}}$	
	2	Horizontal legs of double-angle members with separators or in continuous contact	b/t	$0.47\sqrt{\frac{E}{R_y F_y}}$	
Stiffened Elements	3	Where used in beams or columns as flanges in uniform compression due to flexure or combined axial and flexure (1) Flanges of rectangular HSS[a] (2) Flanges of boxed I-shaped sections (3) Flanges of box sections	b/t	$0.55\sqrt{\frac{E}{R_y F_y}}$	

TABLE A-N10.2.1 (continued)
Limiting Width-to-Thickness Ratios for Structural Steel and Composite Members

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_c	Examples
Stiffened Elements	4	Where used in beams, columns, or links, as webs in flexure, or combined axial and flexure (1) Side plates of boxed I-shaped sections (2) Webs of rectangular HSS[a] (3) Webs of box sections (4) Webs of rolled or built-up I-shaped sections and channels	h/t	$1.56\sqrt{\dfrac{E}{R_yF_y}}$	
	5	Walls of round HSS[a]	D/t	$0.038\dfrac{E}{R_yF_y}$	
Composite Elements	6	Flanges and webs of filled rectangular HSS and box sections[a]	b/t h/t	$1.4\sqrt{\dfrac{E}{R_yF_y}}$	
	7	Walls of filled round HSS sections[a]	D/t	$0.076\dfrac{E}{R_yF_y}$	

[a] The design wall thickness shall be used in the calculations involving the wall thickness of hollow structural sections (HSS), as defined in *Specification* Section B4.2.

TABLE A-N10.2.2
Permissible Ductility Ratio, μ_p, for Design of Structural Elements Subjected to Impactive or Impulsive Loads

Limit State	Permissible Ductility Ratio
Tension[a]	$\mu_p \le 0.25\ \varepsilon_u/\varepsilon_y \le 0.1/\varepsilon_y$ [b]
Flexure[a],[c]	
Steel plates	$\mu_p \le 20$
Open sections such as W, S, and WT	$\mu_p \le 10$
Closed sections such as pipe and box section	$\mu_p \le 20$
structural elements where shear governs design	$\mu_p \le 5$
Compression (applicable when $F_e \ge 4.5F_y$)	$\mu_p = 0.225/(F_y/F_e) \le \varepsilon_{st}/\varepsilon_y$ not to exceed 10 [d]

[a] For net sections with ductile behavior, the plastic resistance shall be based on yielding of the net section. For net sections with either brittle or limited ductile behavior, the structural element's plastic resistance shall be based on yielding of the gross section provided that the net section's tensile rupture based available strength exceeds its gross section's yielding based available strength.

[b] ε_u = strain corresponding to elongation at failure (rupture) using the value corresponding to an 8-in.-long (200 mm) tensile coupon specimen
ε_y = strain corresponding to nominal yield stress = F_y/E

[c] Accompanying compression force, if any, shall be less than the smaller of $0.1F_eA_g$ and $0.1F_yA_g$.

[d] $F_e = \pi^2 E/(L_c/r)^2$; ε_{st} = strain corresponding to the onset of strain hardening using the value corresponding to an 8-in.-long (200 mm) tensile coupon specimen

N10.3. ANALYSIS, DESIGN, AND DETAILING OF SC STRUCTURAL ELEMENTS

1. Compactness Requirements

The boundary region compactness requirements of Appendix N9, Section N9.1.3, shall be satisfied.

2. Local Response Evaluation

The minimum required perforation thickness for SC structural elements subjected to impactive loads shall be determined using project-specific test data or applicable published analytical methods developed from validated test data. In lieu of specific test data or published methods, the minimum required faceplate thickness, $t_{p,min}$, shall be determined as follows:

$$t_{p,min} = 0.066\left[\frac{V_r^2}{d^2\sigma_r}\left(\frac{W_p + W_{cf}}{g}\right)\right],\ \text{in.} \qquad \text{(A-N10-2)}$$

$$t_{p,min} = 458\left[\frac{V_r^2}{d^2\sigma_r}\left(\frac{W_p + W_{cf}}{g}\right)\right],\ \text{mm} \qquad \text{(A-N10-2M)}$$

where,

V_r = residual velocity of a missile passing through concrete, ft/s (m/s)

$$= \sqrt{\left(\frac{W_p}{W_p + W_{cf}}\right)\left(V_i^2 - V_p^2\right)} \qquad \text{(A-N10-3)}$$

V_i = initial (pre-impact) velocity of missile, ft/s (m/s)

V_p = perforation velocity for reinforced concrete section of same thickness, per NEI 07-13, ft/s (m/s)

$$= 1{,}000d\left\{\frac{d}{1.44K_pW_pNK_{psc}^2}\left[2.2 \pm \sqrt{4.84 - 1.2\left(\frac{t_c}{\alpha_p d}\right)}\right]^2\right\}^{5/9} \text{ when } \frac{t_c}{\alpha_p d} \le 2.65 \qquad \text{(A-N10-4a)}$$

$$= 1{,}000d\left[\frac{d}{4K_pW_pNK_{psc}^2}\left(\frac{t_c}{1.29\alpha_p d} - 0.53\right)^2\right]^{5/9} \text{ when } 2.65 < \frac{t_c}{\alpha_p d} < 3.27 \qquad \text{(A-N10-4b)}$$

$$= 1{,}000d\left\{\frac{d}{K_pW_pNK_{psc}}\left[\frac{t_c}{1.29\alpha_p d} - \left(0.53 + K_{psc}\right)\right]\right\}^{5/9} \text{ when } \frac{t_c}{\alpha_p d} \ge 3.27 \qquad \text{(A-N10-4c)}$$

$$= 3.724d\left\{\frac{d}{K_pW_pNK_{psc}^2}\left[2.2 \pm \sqrt{4.84 - 1.2\left(\frac{t_c}{\alpha_p d}\right)}\right]^2\right\}^{5/9} \text{ when } \frac{t_c}{\alpha_p d} \le 2.65 \qquad \text{(A-N10-4aM)}$$

$$= 2.10d\left[\frac{d}{K_pW_pNK_{psc}^2}\left(\frac{t_c}{1.29\alpha_p d} - 0.53\right)^2\right]^{5/9} \text{ when } 2.65 < \frac{t_c}{\alpha_p d} < 3.27 \qquad \text{(A-N10-4bM)}$$

$$= 4.56d\left\{\frac{d}{K_pW_pNK_{psc}}\left[\frac{t_c}{1.29\alpha_p d} - \left(0.53 + K_{psc}\right)\right]\right\}^{5/9} \text{ when } \frac{t_c}{\alpha_p d} \ge 3.27 \qquad \text{(A-N10-4cM)}$$

K_p = strength-dependent concrete penetrability factor

$$= 5.692 \Big/ \sqrt{f_c'} \qquad \text{(A-N10-5)}$$

$$= 14.95 \Big/ \sqrt{f_c'} \qquad \text{(A-N10-5M)}$$

K_{psc} = penetration depth modification factor for SC cross section

$$= 2.073 - 0.661K_p + 0.688\left(\frac{\alpha_p d}{t_c}\right) + 0.835\left(\frac{x_c}{t_c}\right) \qquad \text{(A-N10-6)}$$

N = missile nose shape factor per the modified NDRC formula

= 0.72 for flat-nosed missiles

= 0.84 for blunt-nosed missiles

= 1.00 for spherical-nosed missiles

= 1.14 for sharp-nosed missiles

W_{cf} = weight of the concrete frustum (plug) associated with x_{sc}, the penetration depth, of the impacting missile, lb (N)

W_p = missile weight, lb (N)

$$=\frac{1}{3}\pi\left(\frac{\rho_c}{12^3}\right)(t_c-x_{sc})\left(r_2^2+r_1r_2+r_1^2\right) \text{ when } x_{sc}<t_c \quad \text{(A-N10-7a)}$$

$$=\frac{1}{3}\pi\left(\frac{g\rho_c}{10^9}\right)(t_c-x_{sc})\left(r_2^2+r_1r_2+r_1^2\right) \text{ when } x_{sc}<t_c \quad \text{(A-N10-7aM)}$$

$$= 0 \text{ when } x_{sc} \geq t_c \quad \text{(A-N10-7b)}$$

d = effective diameter of the missile, in. (mm)

f_c' = compressive strength of concrete, ksi (MPa)

g = acceleration due to gravity, in./s^2 (m/s^2)

= 386 in./s^2 (9.81 m/s^2)

r_1 = effective radius of the missile, in. (mm)

r_2 = concrete frustum radius at the inside face of the back faceplate, in. (mm)

$$=r_1+(t_c-x_{sc})\tan\theta \text{ when } x_{sc}<t_c \quad \text{(A-N10-8a)}$$

$$= 0 \text{ when } x_{sc} \geq t_c \quad \text{(A-N10-8b)}$$

t_c = concrete infill thickness, in. (mm)

x_c = concrete penetration depth for the reinforced concrete section of the same thickness as the SC cross section, in. (mm)

$$=\sqrt{4K_pNW_pd\left(\frac{V_i}{1{,}000d}\right)^{1.80}} \text{ when } \frac{x_c}{d}\leq 2.0 \quad \text{(A-N10-9a)}$$

$$=K_pNW_p\left(\frac{V_i}{1{,}000d}\right)^{1.80}+d \text{ when } \frac{x_c}{d}>2.0 \quad \text{(A-N10-9b)}$$

$$=0.511\sqrt{K_pNW_pd\left(\frac{V_i}{d}\right)^{1.80}} \text{ when } \frac{x_c}{d}\leq 2.0 \quad \text{(A-N10-9aM)}$$

$$=0.0652K_pNW_p\left(\frac{V_i}{d}\right)^{1.80}+d \text{ when } \frac{x_c}{d}>2.0 \quad \text{(A-N10-9bM)}$$

x_{sc} = missile penetration depth into the SC cross section, in. (mm)

$$=K_{psc}x_c \quad \text{(A-N10-10)}$$

α_p = missile deformability factor per NEI 07-13

= 0.60 for deformable missiles

$= 1.00$ for rigid missiles

θ = inclination angle of the concrete frustum

$$= \frac{45°}{\left(t_c/d\right)^{1/3}} \tag{A-N10-11}$$

ρ_c = concrete density, lb/ft^3 (kg/m^3)

σ_r = equivalent radial compressive stress in the rear faceplate, based on von Mises yield criterion, ksi (MPa)

$$= 5.1F_y + 101 \text{ when } t_p \geq 0.25 \text{ in.} \tag{A-N10-12a}$$

$$= 3.9F_y + 64 \text{ when } t_p < 0.25 \text{ in.} \tag{A-N10-12b}$$

$$= 5.1F_y + 696 \text{ when } t_p \geq 6 \text{ mm} \tag{A-N10-12aM}$$

$$= 3.9F_y + 441 \text{ when } t_p < 6 \text{ mm} \tag{A-N10-12bM}$$

3. Special Analysis, Design, and Detailing Requirements

Ductility shall be verified in accordance with Section N10.3.4(c) when either of the following conditions is present:

(a) large opening(s), or
(b) small opening(s) with free edge at the opening parameter

> **User Note**: Where possible and practical, an independent impact barrier structure should be provided to spare an SC structural element with either condition (a) or (b) described in Section N10.3.3 from being directly subjected to impulsive or impactive loads.

Bolted attachments to the tension faceplate are permitted if the net section fracture limit state does not control. Except for the case of shop welding associated with the reinforcement around a small opening, welded attachments to the tension faceplate are not permitted in regions that are expected to undergo yielding when subjected to the specified impulsive or impactive loading.

Only yielding shear reinforcement are permitted. Additionally, the available out-of-plane shear strength shall be at least 120% of the out-of-plane shear strength corresponding to the flexure-controlled failure mechanism.

> **User Note**: The out-of-plane shear strength requirement specified in Section N10.3.3 ensures that the SC structural element subjected to impactive or impulsive load will undergo significant inelastic response through flexural yielding, rather than the significantly less ductile failure mechanism associated with the yielding of shear reinforcement.

4. Analysis Requirements for Verification of Structural Element Ductility

The response of SC structural elements subjected to impactive and impulsive loads shall be determined by one of the following three methods:

(a) If the target response remains elastic, the dynamic load effects of the impulsive or impactive loads shall be calculated using the applicable dynamic load factor (DLF). The calculated maximum elastic demands using this method shall not exceed the capacities defined in Appendix N9, Section N9.3.
(b) If the target response is in the inelastic range, use of a simplified single-degree-of-freedom analysis of the target, using either bilinear or multi-linear resistance function, is permitted. The presence of concurrent membrane forces, if any, shall be accounted for when developing the resistance function. The calculated maximum support rotation using this method shall not exceed 6-deg (0.105 rad).
(c) Alternatively, if the target response is in the inelastic range, use of a detailed nonlinear and inelastic finite element analysis is permitted for direct determination of plastic strains. The maximum plastic strain using this method shall not exceed 0.05 in./in. (mm/mm) for the faceplates and 0.005 in./in. (mm/mm) for ties classified as yielding shear reinforcement.

User Note: Analysis and design of SC structural elements subjected to impactive or impulsive loads requires subject matter expertise. In particular, implementation of option (c) is more involved because it requires accurate determination of the structural element's stress-strain curve and its maximum response. Peer review by independent subject matter expert(s) is recommended if option (c) is implemented.

The method per option (b) is easier to implement because it involves a simplified bilinear (or multilinear) resistance function of the structural element's load-displacement behavior that is based on equivalent single-degree-of-freedom model (accordingly, the permissible plastic rotation limit has been conservatively specified). This method is based on similar provisions in UFC 3-340-02 (DOD, 2008), which requires determination of the structural element's resistance function by using its nominal yield strength times the dynamic increase factor and applicable strain-hardening effect. As defined and illustrated in UFC 3-340-02 (DOD, 2008), the effective yield point is taken as the intersection point of the line representing the initial equivalent stiffness with the horizontal line representing the plastic behavior (see commentary for further discussion). The associated effective yield displacement is used for implementation of option (b).

SC basemats subjected to impulsive or impactive loads shall be evaluated using option (c).

For all methods, the associated connections shall be designed such that their available strengths are greater than R_y times the nominal strength for LRFD and R_y/1.5 times the nominal strength for ASD of the connected structural element, where the R_y value corresponds to the material used in the connected structural element and is obtained from *Seismic Provisions* Table A3.2.