ANSI/AISC N690-18
An American National Standard

Specification for Safety-Related Steel Structures for Nuclear Facilities

June 28, 2018

Supersedes the *Specification for Safety-Related Steel Structures for Nuclear Facilities* dated January 31, 2012 including Supplement No. 1 dated August 11, 2015 and all previous versions

Approved by the Committee on Specifications

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Stronger.
Steel.

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PREFACE

(This Preface is not part of ANSI/AISC N690-18, but is included for informational purposes only.)

The AISC *Specification for Safety-Related Steel Structures for Nuclear Facilities*, hereafter referred to as the Nuclear Specification, addresses the design, fabrication and erection of safety-related steel structures for nuclear facilities. This document uses the 2016 AISC *Specification for Structural Steel Buildings*, hereafter referred to as the *Specification*, as the baseline document and modifies the specific portions of the *Specification* to make it applicable to the design, fabrication and erection of safety-related steel structures for nuclear facilities. Nonmandatory User Notes and Commentary provide additional guidance and background for the Nuclear Specification provisions, and the user is encouraged to consult them.

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

This revision incorporates Supplement No. 1 to the 2012 *Specification for Safety-Related Steel Structures for Nuclear Facilities* and also incorporates revisions required for consistency with the 2016 *Specification for Structural Steel Buildings*.

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

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SYMBOLS

The symbols listed below shall be used in addition to or as replacements for those in the AISC *Specification for Structural Steel Buildings*. The section or table number in the right-hand column refers to where the symbol is first used.

Symbol	Definition	Section
A_c	Area of concrete infill per unit width, in.^{2}/ft (mm^2/m)	App. N9.2.2
A_s	Gross area of faceplates per unit width, in.^{2}/ft (mm^2/m)	App. N9.1.1
A_s^F	Gross cross-sectional 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_m	Maximum displacement from analysis (in accordance with Section N9.1.6c), in. (mm)	App. N9.1.6b
D_y	Effective yield displacement, in. (mm)	App. N9.1.6b
E_m	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, as defined in the U.S. Nuclear Regulatory Commission document, "Earthquake Engineering Criteria for Nuclear Power Plants," Appendix S, 10CFR50, or as specified by the AHJ	NB2.2
E_s	Loads generated by safe shutdown or design basis earthquake, as defined in the U.S. Nuclear Regulatory Commission document, "Earthquake Engineering Criteria for Nuclear Power Plants," Appendix S, 10CFR50, or as specified by the AHJ	NB2.3
E_s	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa) for carbon steel = 28,000 ksi (193 000 MPa) for stainless steel	App. N9.1.3
EI_{eff}	Effective flexural stiffness for analysis of SC walls per unit width, kip-in.^{2}/ft (N-mm^2/m)	App. N9.2.4
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_{ny}	Nominal yield strength of tie, kips (N)	App. N9.1.5a
F_{nr}	Nominal rupture strength of tie, or nominal strength of associated connection, whichever is smaller, kips (N)	App. N9.1.5a
F_{req}	Required tensile strength for individual ties, kips (N)	App. N9.1.5b
F_t	Nominal tensile strength of ties, kips (N)	App. N9.3.5

Symbol	Definition	Section
G	Shear modulus of elasticity of steel. = 11,200 ksi (77 200 MPa) for carbon steel = 10,800 ksi (74 500 MPa) for stainless steel	App. N9.2.2
GA_{eff}	Effective in-plane shear stiffness per unit width, kip/ft (N/m)	App. N9.2.2
GA_{uncr}	In-plane shear stiffness of uncracked composite SC panel section per unit width, kip/ft (N/m)	App. N9.2.2
G_c	Shear modulus of concrete, ksi (MPa)	App. N9.2.2
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 of faceplates per unit width (corresponding to the condition when concrete is fully cracked), in.^{4}/ft (mm^4/m)	App. N9.2.2
L	Live load due to occupancy and moveable equipment, including impact	NB2.1
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. N9.3.3
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
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 steel anchor, kips (N)	App. N9.1.4a
Q_{cv}^{avg}	Weighted average of available interfacial shear strength of ties and steel anchors while accounting for their respective tributary areas and numbers, kips (N)	App. N9.3.6a
R	Rain load	NB2.1
R_a	Pipe and equipment reactions generated by postulated accident, including R_o	NB2.4
R_o	Pipe reactions during normal operating, start-up or shutdown conditions, based on most critical transient or steady-state condition	NB2.1
S	Snow load as stipulated in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7) for Risk Category IV facilities.	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

Symbol	Definition	Section
$S_{r,min}$	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_{ry}	Required membrane axial strength per unit width in direction y, 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'_{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'_{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
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
T_a	Thermal loads generated by 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 for each notional half 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 strength per unit width of SC panel section, kip/ft (N/m)	App. N9.1.4b
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. N9.3.5a
V_{ni}	Nominal in-plane shear strength per unit width of SC panel section, kip/ft (N/m)	App. N9.3.6
V_{no}	Nominal out-of-plane shear strength per unit width of SC panel section, kip/ft (N/m)	App. 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 or ASD load combinations, kip/ft (N/m)	App. N9.3.6a
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 as stipulated in ASCE/SEI 7 for Risk Category IV facilities, or as specified by the AHJ	NB2.2

Symbol	Definition	Section
W_t	Loads generated by the specified design (basis) tornado, including wind pressures, pressure differentials, and tornado-borne missiles, as defined in U.S. Nuclear Regulatory Commission Standard Review Plan 3.3.2 (NUREG-0800), or as specified by the AHJ	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 broken high-energy pipe during the postulated accident	NB2.4
b	Largest unsupported length of faceplate between rows of steel anchors or ties, in. (mm)	App. N9.1.3
c_2	Calibration constant for determining effective flexural stiffness	App. N9.2.2
c_m	Specific heat used in elastic finite element analysis of SC panel section, Btu/lb-°F (J/kg-°C)	App. N9.2.3
f_w	Faceplate waviness	NM2.7
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
l	Unit width, 12 in./ft (1000 mm/m)	App. N9.1.4b
n	Modular ratio of steel and concrete	App. N9.2.2
s	Spacing of steel anchors, in. (mm)	App. N9.3.6a
s_{tl}	Spacing of shear reinforcement along the direction of one-way shear, in. (mm)	App. N9.1.5b
$s_{t,min}$	Minimum tie spacing, in. (mm)	NM2.7
s_{tt}	Spacing of shear reinforcement transverse to the direction of one-way shear, in. (mm)	App. N9.1.5b
t_c	Concrete infill thickness, in. (mm)	App. N9.2.2
t_m	Model section thickness used in elastic finite element analysis of SC panel section, in. (mm)	App. N9.2.3
t_p	Thickness of faceplate, in. (mm)	NM2.7
t_{sc}	SC section thickness, in. (mm)	App. N9.1.1
ΔT_{savg}	Average of the maximum surface temperature increases for the faceplates due to accident thermal conditions, °F (°C)	App. N9.2.2
ΔT_{sg}	Maximum temperature difference between faceplates due to accident thermal conditions in °F (°C)	App. N9.2.4
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

Symbol	Definition	Section
α_m	Thermal expansion coefficient used in the elastic finite element analysis of SC panel section, °F^{-1} (°C^{-1})	App. N9.2.3
α_s	Thermal expansion coefficient of faceplate, °F^{-1} (°C^{-1})	App. N9.2.4
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	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
κ	Calibration constant for determining in-plane shear strength	App. N9.3.4
κ_m	Thermal conductivity used in elastic finite element analysis of SC panel section, Btu/ft-sec-°F (W/m-°C)	App. N9.2.3
ε_{st}	Strain corresponding to the onset of strain hardening	Table NB3.1
ε_u	Strain corresponding to elongation at failure (rupture)	Table NB3.1
ε_y	Strain corresponding to nominal yield stress	Table NB3.1
μ_p	Permissible ductility ratio	NB3.14
μ_r	Required ductility ratio	NB3.14
ν_m	Poisson's ratio used in elastic finite element analysis	App. N9.2.3
ρ	Reinforcement ratio	App. N9.1.1
$\bar{\rho}$	Strength-adjusted reinforcement ratio	App. N9.2.2
ρ'	Stiffness-adjusted reinforcement ratio	App. N9.2.2
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*.

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

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

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 NB3.2. 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 (SC) wall used for elastic finite element analysis.

Effective in-plane shear stiffness. Cracked transformed shear stiffness of the SC wall used for elastic finite element analysis.

Effective SC stiffness. Effective stiffness of the SC 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 shall be a licensed professional engineer, qualified to fulfill the assigned responsibility.

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

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

Interior region. Region of SC wall 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 SC walls 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 reinforcement. Ties that do not meet the requirements of yielding shear reinforcement.

Nonyielding steel anchor. Anchors that do not meet the requirements of yielding steel anchors.

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 SC panel section consisting of one faceplate and half the concrete thickness.

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

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

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

Panel section. The extent of the SC wall over which the demands are averaged to calculate the required strengths. The extent or size of the panel section is provided in Section N9.2.5.

Permissible ductility ratio. Ratio of permitted inelastic strain (or deflection) to the strain (or deflection) at the effective yield point on the idealized bilinear elastic-plastic stress-strain (or 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 specification.

Quality assurance inspector (QAI). Individual(s) designated to independently provide quality assurance inspection for the work being performed. The QAI is permitted to be employed by the EOR, detailer, fabricator, erector, contractor and/or constructor.

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

Quality control inspector (QCI). Individual(s) designated to provide quality control inspection for the work being performed. The QCI is permitted to be employed by the fabricator, erector, contractor and/or constructor.

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.

Ribs. Steel section used to increase faceplate stiffness and strength to handle rigging and construction loads (for example, wet concrete pressure).

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.

Steel-plate composite (SC) wall. A SC wall consists of two steel plates (faceplates) composite with structural concrete between them, where the faceplates are anchored to concrete using steel anchors and connected to each other using ties.

Section thickness. The total thickness of the SC panel section.

Small opening. An opening in the SC wall 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, USNRC Regulatory Guide 1.76).

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

Tie. Structural components such as steel shapes, frames or bars that tie the two faceplates of an SC wall together at regular intervals. The ties provide structural integrity by preventing section splitting and by anchoring the faceplates to concrete after concrete hardening. Ties also serve as out-of-plane shear reinforcement.

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 0.80 times the nominal rupture strength and 0.80 times the nominal strength of the associated connection.

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 Drawings and Specifications
NA5. Quality Assurance

NA1. SCOPE

Replace section with the following:

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

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

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

The Nuclear Specification includes the list of additional Symbols, additional Glossary terms, Chapters NA through NN, and Appendices N1 through N9. 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.

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 steel-plate composite (SC) walls 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 AISI *North American Specification for the Design of Cold-Formed Steel Structural Members* is recommended, incorporating the loads and load combinations delineated in Section NB2 of the Nuclear Specification.

NA2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Add the following:

American Iron and Steel Institute (AISI)

AISI S100-16 *North American Specification for the Design of Cold-Formed Steel Structural Members*

Crane Manufacturers Association of America

CMAA-70 "Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes," 2015

U.S. Nuclear Regulatory Commission

NUREG-0800 *Standard Review Plan*, July 2014

Regulatory Guide 1.54 "Service Level I, II, and III Protective Coatings Applied to Nuclear Power Plants," October 2010

U.S. Code of Federal Regulations

Title 10 of the *Code of Federal Regulations*, Part 50 (10CFR50), Appendix B and Appendix S, 2007

Title 10 of the *Code of Federal Regulations*, Part 830, Subpart A, Quality Assurance Requirements (to be used for Department of Energy Nuclear Facilities), 2011

Title 10 of the *Code of Federal Regulations*, Part 100 (10CFR100), Reactor Site Criteria, 2016

Electric Power and Research Institute

EPRI NP-5380 *Visual Weld Acceptance Criteria, Volume 1, Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants (NCIG-01, Revision 2): Final Report*, 1987

EPRI NP-5380 *Visual Weld Acceptance Criteria, Volume 2, Sampling Plan for Visual Reinspection of Welds (NCIG-02, Revision 2): Final Report*, 1987

EPRI NP-5380 *Visual Weld Acceptance Criteria, Volume 3, Training Manual for Inspectors of Structural Welds at Nuclear Power Plants Using the Acceptance Criteria of NCIG-01 (NCIG-03, Revision 1): Final Report*, 1987

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 360-16 *Specification for Structural Steel Buildings*

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

ANSI/AISC N690-12 *Specification for Safety-Related Steel Structures for Nuclear Facilities*

ANSI/AISC N690s1-15 *Specification for Safety-Related Steel Structures for Nuclear Facilities, Including Supplement No. 1*

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

ANSI/ASCE 8-02 *Specification for the Design of Cold-Formed Stainless Steel Structural Members*

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

ASME NQA-1-2015 “Quality Assurance Requirements for Nuclear Facility Applications”

Boiler and Pressure Vessel Code Section III, Div. 1, 2015

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

A20/A20M-15 *Standard Specification for General Requirements for Steel Plates for Pressure Vessels*

A27/A27M-17 *Standard Specification for Steel Castings, Carbon, for General Application*

A106/A106M-15 *Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service*

A148/A148M-15a *Standard Specification for Steel Castings, High Strength, for Structural Purposes*

A217/A217M-14 *Standard Specification for Steel Castings, Martensitic Stainless and Alloy, for Pressure-Containing Parts, Suitable for High-Temperature Service*

A240/A240M-16a *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-17 *Standard Specification for Seamless, Welded, and Heavily Cold Worked Austenitic Stainless Steel Pipes*

A320/A320M-17a *Standard Specification for Alloy-Steel Bolting Materials for Low-Temperature Service*

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

A479/A479M-17 *Standard Specification for Stainless Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels*

A515/A515M-10(2015) *Standard Specification for Pressure Vessel Plates, Carbon Steel, for Intermediate- and Higher-Temperature Service*

A516/A516M-10(2010) *Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service*

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

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

A554-16 *Standard Specification for Welded Stainless Steel Mechanical Tubing*

A564/A564M-13 *Standard Specification for Hot-Rolled and Cold-Finished Age-Hardening Stainless Steel Bars and Shapes*

A577/A577M-90(2012) *Standard Specification for Ultrasonic Angle-Beam Examination of Steel Plates*

A578/A578M-07(2012) *Standard Specification for Straight-Beam Ultrasonic Examination of Rolled Steel Plates for Special Applications*

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

A738/A738M-12a *Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel, for Moderate and Lower Temperature Service*

A770/A770M-03(2012)E1 *Standard Specification for Through-Thickness Tension Testing of Steel Plates for Special Applications*

A1008/A1008M-15 *Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability*

D3843-00 (2008) *Standard Practice for Quality Assurance for Protective Coatings Applied to Nuclear Facilities*

F606/F606M-14a *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, Direct Tension Indicators, and Rivets*

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

AWS A5.4/A5.4M:2012 *Specification for Stainless Steel Electrodes for Shielded Metal Arc Welding*
AWS A5.9/A5.9M:2012 *Specification for Bare Stainless Steel Welding Electrodes and Rods*
AWS D1.4/D1.4M:2011 *Structural Welding Code—Reinforcing Steel*
AWS D1.6/D1.6M:2007 *Structural Welding Code—Stainless Steel*
AWS D1.8/D1.8M:2016 *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 A20/A20M. The CVN impact test shall be conducted at a temperature that is at least 30°F (17°C) below the lowest anticipated service temperature of the structural component being evaluated. The acceptance criteria shall be that the material withstand not less than the energy values (average of three specimens value and individual specimen value) indicated in Table NA3.1, in addition to satisfying the applicable ASTM standard.

User Note: For structures or structural components subject to impactive and/or impulsive loads, the lowest anticipated service temperature is the minimum service temperature corresponding to the time when any of the applicable postulated sudden loading events can occur.

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 30°F (17°C) below the lowest anticipated service temperature, the component may be fabricated from plate or bar provided all requirements (CVN and others) applicable to the fabricated shape are met.

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.

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
Equal to or less than 36 ksi (250 MPa)	15 ft-lb (20 J)	10 ft-lb (14 J)
Greater than 36 ksi (250 MPa), less than 44 ksi (300 MPa)	20 ft-lb (27 J)	15 ft-lb (20 J)
Equal to or greater than 44 ksi (300 MPa)	30 ft-lb (41 J)	25 ft-lb (34 J)

1a. ASTM Designations

Modify this section as follows:

(b) Hollow structural sections (HSS)

Add the following:

ASTM A106/A106M
ASTM A312/A312M
ASTM A554

(c) Plates

Add the following:

ASTM A240/A240M
ASTM A515/A515M
ASTM A516/A516M
ASTM A537/A537M Class 1 and Class 2
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

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 not addressed by ANSI/ASCE 8, refer to AISC Design Guide 27, *Structural Stainless Steel*. Additional requirements for stainless steel plates used in steel-plate composite walls 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.

1b. Unidentified Steel

Replace section with the following:

Unidentified steel shall not be used.

1c. 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. A plan shall be developed 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 ¾ in. (19 mm).

1d. 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. A plan shall be developed 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.

2. Steel Castings and Forgings

Replace section with the following:

Steel castings shall conform to ASTM A27/A27M, ASTM A148/A148M, ASTM A216/A216M, or ASTM A217/A217M. Steel forgings shall conform to ASTM A668/A668M.

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 A5.4/A5.4M
AWS A5.5/A5.5M
AWS A5.9/A5.9M
AWS A5.17/A5.17M
AWS A5.18/A5.18M
AWS A5.20/A5.20M
AWS A5.22/A5.22M
AWS A5.23/A5.23M
AWS A5.25/A5.25M
AWS A5.26/A5.26M
AWS A5.28/A5.28M
AWS A5.29/A5.29M
AWS A5.32/A5.32M
AWS A5.36/A5.36M

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 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 DRAWINGS AND SPECIFICATIONS

Replace section with the following:

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

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 plans for the structural elements shall identify those elements or systems that are deemed safety-related by the engineer of record.

> **User Note:** The *Code of Standard Practice* uses the term “design documents” in place of “design drawings” to generalize the term and to reflect both paper drawings and electronic models. Similarly, “fabrication documents” is used in place of “shop drawings,” and “erection documents” is used in place of “erection drawings.” The use of “drawings” in this standard is not intended to create a conflict.

The construction specification 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

Add the following section:

NA5. QUALITY ASSURANCE

A quality assurance program covering safety-related steel structures shall be developed prior to design or construction, as applicable. The general requirements and guidelines for establishing and executing the quality assurance program during the design and construction phases of nuclear facilities are those established by 10CFR50, Appendix B, for Nuclear Power Stations, and as outlined in Chapter NN of the Nuclear Specification.

Calculations pertinent to the design shall be documented and shall include a statement of the applicable design criteria. Calculations shall be performed in accordance with ASME NQA-1, Requirement 3, "Design Control," or other applicable standards approved by the authority having jurisdiction (AHJ). Activities involving specification, design, calculations, drawings, fabrication and erection are subject to quality assurance requirements. Computer programs used in analysis and design shall likewise be covered by a quality assurance program, as provided by ASME NQA-1, Subpart 2.7, "Quality Assurance Requirements for Computer Software for Nuclear Facility Applications."

User Note: 10CFR50, Appendix B, and 10CFR830, Subpart A, provide regulations for quality assurance (QA) and quality control (QC). The requirements of Chapter NN are aimed to assist the user in developing a QA/QC program that will satisfy the regulations.

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:

NB1. General Provisions
NB2. Loads and Load Combinations
NB3. Design Basis
NB4. Member Properties
NB5. Fabrication and Erection
NB6. Quality Control and Quality Assurance
NB7. Evaluation of Existing Structures

NB2. LOADS AND LOAD COMBINATIONS

Replace section with the following:

1. Normal Loads

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

D = 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 as stipulated in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7) for Risk Category IV facilities
T_o = thermal effects and loads during normal operating, start-up or shutdown conditions, based on the most critical transient or steady-state condition

2. Severe Environmental Loads

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

E_o = where required as part of the design basis, loads generated by the operating basis earthquake (OBE) as defined in 10CFR50, Appendix S, or as specified by the AHJ

W = wind load as stipulated in ASCE/SEI 7 for Risk Category IV facilities, or as specified by the AHJ

3. Extreme Environmental Loads

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

E_s = loads generated by the safe shutdown, or design basis earthquake, as defined in 10CFR50, Appendix S, or as specified by the AHJ

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

4. Abnormal Loads

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

P_a = 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 critical structural effect may occur when one or more loads are not acting. The following load combinations shall be investigated:

5a. Normal Load Combinations

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

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

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

5b. Severe Environmental Load Combinations

$$1.2(D+F+R_o)+W+0.8L+1.6H+0.5\left(L_r \text{ or } S \text{ or } R\right)+T_o+C \quad \text{(NB2-4)}$$
$$1.2(D+F+R_o)+1.6E_o+0.8L+1.6H+0.2\left(L_r \text{ or } S \text{ or } R\right)+T_o+C \quad \text{(NB2-5)}$$

5c. Extreme Environmental and Abnormal Load Combinations

$$D+0.8L+C+T_o+R_o+E_s+F+H \quad \text{(NB2-6)}$$
$$D+0.8L+T_o+R_o+W_t+F+H \quad \text{(NB2-7)}$$
$$D+0.8L+C+1.2P_a+R_a+T_a+F+H \quad \text{(NB2-8)}$$
$$D+0.8L+\left(P_a+R_a+T_a\right)+\left(Y_r+Y_j+Y_m\right)+0.7E_s+F+H \quad \text{(NB2-9)}$$

5d. Other Considerations

> **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, etc.). Structural analysis for 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.

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

(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 appropriately considered for steel structure components subjected to these loads. Any fluid structure interaction associated with these hydrodynamic loads and those from the postulated seismic loads shall be taken into account.

(8) In Load Combination NB2-6, the load C is permitted to be waived, provided it can be demonstrated that the probability of E_s and C occurring at the same time is less than 1×10^{-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 most critical structural effects may occur when one or more loads are not acting. The following load combinations shall be investigated:

6a. Normal Load Combinations

$$D + L + R_o + F + H + T_o + C \quad \text{(NB2-10)}$$
$$D + (L_r \text{ or } S \text{ or } R) + R_o + F + H + T_o + C \quad \text{(NB2-11)}$$
$$D + F + 0.75L + 0.75H + 0.75(L_r \text{ or } S \text{ or } R) + T_o + C \quad \text{(NB2-12)}$$

6b. Severe Environmental Load Combinations

$$D + R_o + F + 0.6W + 0.75(L + H) + C + 0.75(L_r \text{ or } S \text{ or } R) + T_o \quad \text{(NB2-13)}$$
$$D + R_o + F + E_o + 0.75(L + H) + C + 0.75(L_r \text{ or } S \text{ or } R) + T_o \quad \text{(NB2-14)}$$

6c. Extreme Environmental and Abnormal Load Combinations

$$D + L + C + R_o + T_o + E_s + F + H \quad \text{(NB2-15)}$$
$$D + L + R_o + T_o + W_t + F + H \quad \text{(NB2-16)}$$
$$D + L + C + P_a + R_a + T_a + F + H \quad \text{(NB2-17)}$$
$$D + L + P_a + R_a + T_a + Y_r + Y_j + Y_m + 0.7E_s + F + H \quad \text{(NB2-18)}$$

6d. Other Considerations

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, etc.). Structural analysis for 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.

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

(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 appropriate load combinations stipulated in Section NB2.

Design by elastic, inelastic or plastic analysis is permitted. Provisions for inelastic and plastic analysis are as stipulated in Appendix N1, Section N1.3, Design by Inelastic Analysis.

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

> **User Note:** Values for the reduction in material properties of structural steels exposed to elevated temperatures can be found in resources such as the *Structural Alloys Handbook*, published by Battelle, Columbus, OH, and in the ASME *Boiler and Pressure Vessel Code*, Section II, Part D, Material Properties. Sustained temperature above 700°F (370°C) may subject the material to creep rupture effects that need to be considered in the design. Properties for fire conditions of commonly used structural steels are tabulated in *Specification* Appendix 4, Table A-4.2.1.

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. Design Based on Ductility and Local Effects

In Load Combinations NB2-7 through NB2-9 of Section NB2.5, and in Load Combinations NB2-16 through NB2-18 of Section NB2.6, it is permitted to determine the load effects for impactive or impulsive forces using inelastic analysis. Design adequacy of members subjected to these load effects shall be assessed by using one of the following two options:

(a) Use the member's stress-strain (or load-deflection) curve for performing an inelastic analysis to demonstrate that the calculated maximum inelastic strain (or deflection) is less than or equal to one-half of the strain (or deflection) corresponding to the onset of plastic instability, or

(b) Use the member's idealized bilinear (or multilinear) elastic-plastic stress-strain (or load-deflection) curve for performing an inelastic analysis to demonstrate that the calculated value of the required ductility ratio, μ_r, is less than or equal to the applicable value of the permissible ductility ratio, μ_p, provided in Table NB3.1.

For both options (a) and (b), 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 member, where the R_y value corresponds to the material used in the connected member and is obtained from *Seismic Provisions* Table A3.1.

The limiting width-to-thickness ratios for compression elements in members subject to flexure or compression shall not exceed λ_r as given in Table NB3.2. Members in flexure only, or combined flexure and compression, shall conform to the lateral bracing requirements of *Specification* Appendix 1, Section 1.3.2c.

User Note: Analysis and design of members subjected to impulsive or impactive loads requires subject matter specialty. In particular, implementation of option (a) is more involved because it requires rigorous determination of the member's stress-strain curve (or its load-deflection curve, as appropriate), its maximum resistance, and strain (or deflection) level corresponding to the onset of plastic instability. Peer review by independent subject matter expert(s) is recommended if option (a) is implemented.

The method per option (b) is easier to implement because it is based on bilinear (or multilinear) elastic-plastic idealization of the member's stress-strain (or load-deflection) behavior (accordingly, the permissible ductility ratios in Table NB3.1 have been conservatively specified). This method is based on determination of the member's idealized plastic resistance level using its nominal yield strength times the dynamic increase factor. The effective yield point is taken as the intersection point of the line representing (the initial stiffness based) elastic behavior with the horizontal line representing the plastic behavior (see commentary for further discussion and illustration). The resulting effective yield strain (or deflection) is used for implementation of option (b).

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

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

Steel-plate composite (SC) walls shall be designed for impactive and impulsive loads in accordance with Appendix N9, Section N9.1.6.

TABLE NB3.1
Permissible Ductility Ratio, μ_p, for Design of Structural Members 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 (W, S, WT, etc.)	$\mu_p \le 10$
Closed sections (pipe, box section, etc.)	$\mu_p \le 20$
Members 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 member'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) 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] $F_e = \pi^2E/(L_c/r)^2$; ε_{st} = strain corresponding to the onset of strain hardening

User Note: The elongation values for 8-in.-long (200 mm) specimens are readily available from the applicable ASTM material standards.

NB5. FABRICATION AND ERECTION

Replace section with the following:

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

NB6. QUALITY CONTROL AND QUALITY ASSURANCE

Replace section with the following:

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

NB7. EVALUATION OF EXISTING STRUCTURES

Replace section with the following:

Provisions for the evaluation of existing structures shall conform to the requirements of Appendix N5, Evaluation of Existing Structures.

TABLE NB3.2
Limiting Width-to-Thickness Ratios for Compression Elements per Section NB3.14

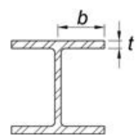
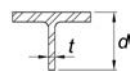
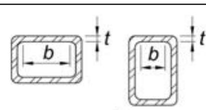
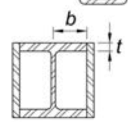
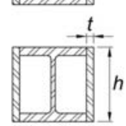
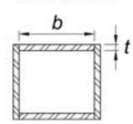
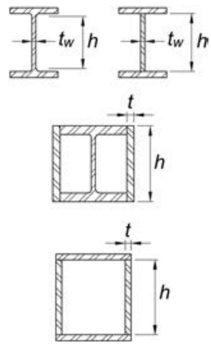
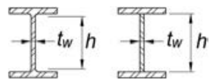
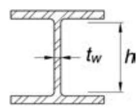
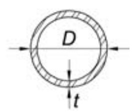
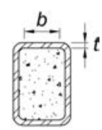
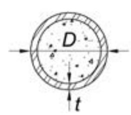
	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r	Example
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees Legs of single angles or double-angle members with separators Outstanding legs of pairs of angles in continuous contact	b/t	$0.30\sqrt{E/F_y}$	
	Flanges of H-pile sections	b/t	$0.45\sqrt{E/F_y}$	
	Stems of tees	d/t	$0.30\sqrt{E/F_y}$ [a]	
Stiffened Elements	Walls of rectangular HSS	b/t	$0.55\sqrt{E/F_y}$ [b]	
	Flanges of boxed I-shaped sections and built-up box sections	b/t		
	Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces	h/t		

TABLE NB3.2 (continued)
Limiting Width-to-Thickness Ratios for Compression Elements per Section NB3.14

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r	Example
Stiffened Elements	Webs of rolled or built-up I-shaped sections used as beams or columns Side plates of boxed I-shaped sections used as beams or columns Webs of built-up box sections used as beams or columns	h/t_w h/t h/t	For $C_a \le 0.125$ $2.45\sqrt{E/F_y}\,(1-0.93C_a)$ For $C_a > 0.125$ $0.77\sqrt{E/F_y}\,(2.93-C_a)$ $\ge 1.49\sqrt{E/F_y}$ where $C_a = \dfrac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \dfrac{\Omega_c P_a}{P_y}$ (ASD)	
	Webs of rolled or built-up I shaped sections used as diagonal braces	h/t_w	$1.49\sqrt{E/F_y}$	
	Webs of H-pile sections	h/t_w	$0.94\sqrt{E/F_y}$	
	Walls of round HSS	D/t	$0.038\,E/F_y$	
Composite Elements	Walls of rectangular filled composite members	b/t	$1.40\sqrt{E/F_y}$	
	Walls of round filled composite members	D/t	$0.076\,E/F_y$	

[a] For tee-shaped compression members, the limiting width-to-thickness ratio for highly ductile members for the stem of the tee is permitted to be increased to $0.38\sqrt{E/F_y}$ if either of the following conditions are satisfied:
(1) Buckling of the compression member occurs about the plane of the stem.
(2) The axial compression load is transferred at end connections to only the outside face of the flange of the tee, resulting in an eccentric connection that reduces the compression stresses at the tip of the stem.

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

CHAPTER NC

DESIGN FOR STABILITY

Modify Chapter C of the Specification as follows.

Add the following item to the list of five (a through e) in the first paragraph of Section C1:

(f) and the effects of elevated temperatures.

CHAPTER ND

DESIGN OF MEMBERS FOR TENSION

No changes to Chapter D of the Specification.

CHAPTER NE

DESIGN OF MEMBERS FOR COMPRESSION

No changes to Chapter E of the Specification.

CHAPTER NF

DESIGN OF MEMBERS FOR FLEXURE

No changes to Chapter F of the Specification.

CHAPTER NG

DESIGN OF MEMBERS FOR SHEAR

No changes to Chapter G of the Specification.

CHAPTER NH

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

No changes to Chapter H of the Specification.

CHAPTER NI

DESIGN OF COMPOSITE MEMBERS

Modify Chapter I of the Specification as follows.

Replace "ACI 318, Chapter 17" ***with*** "ACI 349 or ACI 349M, Appendix D" ***and replace*** "ACI 318" ***with*** "ACI 349 or ACI 349M."

CHAPTER NJ

DESIGN OF CONNECTIONS

Modify Chapter J of the Specification as follows.

NJ1. GENERAL PROVISIONS

Modify section as follows.

Replace Section J1.10 with the following:

10. Rivets

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

NJ2. WELDS

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 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, subject to tension normal to the effective area, unless the joints are designed using the nominal strength and resistance factor or safety factor, as applicable, for a partial-joint-penetration groove weld.

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

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 AND THREADED PARTS

Modify section as follows.

1. 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

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

(5) Other connections stipulated on the design documents

10. Bearing and Tearout Strength at Bolt Holes

Replace paragraph (a) with the following:

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

(1) Bearing

$$R_n = 2.4dtF_u \tag{J3-6a}$$

(2) Tearout

$$R_n = 1.2l_c tF_u \tag{J3-6b}$$

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

Add the following new section:

13. 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:

L1. General Provisions
L2. Deflections
L3. Drift
L4. Vibration
L5. Wind-Induced Motion
L6. Thermal Expansion and Contraction
L7. 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 appropriate load combinations stipulated in Section NB2 and the applicable Appendices.

CHAPTER NM

FABRICATION AND ERECTION

Modify Chapter M of the Specification as follows.

NM1. SHOP AND ERECTION DRAWINGS

Replace section with the following:

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

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

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

NM2. FABRICATION

1. Cambering, Curving and Straightening

Modify section to read as follows:

Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature and straightness. The temperature of heated areas shall not exceed 1,100°F (590°C) for ASTM A514/A514M and ASTM A852/A852M steel nor 1,200°F (650°C) for other carbon steels. The temperature of heated areas for ferritic, martensitic or duplex stainless steels shall not exceed 600°F (320°C). The temperature of heated areas for austenitic stainless steel shall not exceed 800°F (430°C). The temperature of heated areas for precipitation hardening stainless steel shall not exceed the ageing temperature.

2. Thermal Cutting

Modify first paragraph to read as follows:

Thermally cut edges shall meet the requirements of AWS D1.1/D1.1M, Sections 5.14.5.2, 5.14.8.3 and 5.14.8.4 with the exception that thermally cut free edges that will not be subject to fatigue shall be free of round-bottom gouges greater than 3⁄16 in. (5 mm) deep and sharp V-shaped notches. Gouges deeper than 3⁄16 in. (5 mm) and notches shall be removed by grinding or repaired by welding. Notches or gouges greater than 3⁄16 in. (5 mm) up to 3⁄8 in. (10 mm) deep that remain from cutting shall be removed by grinding at a slope of 1 to 2½. 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 J2.

> **User Note:** Welder qualifications tests on plate defined in AWS D1.1/D1.1M, clause 4, and AWS D1.6/D1.6M, clause 4, are appropriate for welds connecting plates, shapes or HSS to other plates, shapes or rectangular HSS.

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

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

Welds 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 6, 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 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 Section J3.3.

(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) Wall Panels
Dimensional tolerances of SC wall panels 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 tie-bars located closest to the free edges of SC panels.

(3) 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 wall panels.

(4) 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.

(5) 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 wall panels, 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.

Dimensional tolerances for erected modules before concrete placement 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 wall.

Dimensional tolerances for SC modules 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:

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

where

s = spacing of the steel anchors, in. (mm)

$s_{t,min}$ = minimum tie spacing, in. (mm)

t_p = thickness of faceplate, in. (mm)

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

14. Commercial Grade Dedication

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

15. Identification of Steel

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

The material 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, in such a manner that the material test report is obtainable throughout the service life of 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 in such a manner that the material test report is obtainable throughout the service life of 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 in such a manner that the material test report is obtainable throughout the service life of the structure.

NM3. SHOP PAINTING

4. Finished Surfaces

Replace section with the following:

Except for stainless steels, machine-finished surfaces shall be protected against corrosion by a rust-inhibitive coating that 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 does 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* 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 must satisfactorily resist impact and hydrostatic loads of fluid concrete during placement of concrete within the structure. Bracing shall be left in place as long as required for safety.

Add the following new sections:

7. Tolerances for Cranes

7a. Tolerances for Crane Column Base Lines

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

7b. Tolerances for Crane Runway Girders

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

7c. Tolerances for Crane Rails

Center-to-center distances of crane rails and the straightness of crane rails shall meet the tolerances prescribed by “Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes” (CMAA-70). Vertical misalignment of crane rails measured at centerlines of columns shall meet the tolerances prescribed by CMAA-70. For polar cranes, the 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 ¾ 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 NA5 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 QA program. 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 in the Nuclear Specification, and steel deck manufacturers, are not considered fabricators or erectors.

NN2. FABRICATOR AND ERECTOR QUALITY ASSURANCE PROGRAM

The fabricator and erector shall establish, maintain and document procedures and perform inspections to ensure that their work is performed in accordance with the established quality assurance program, the appropriate elements of the standard, the Nuclear Specification, and the construction documents. The quality assurance program shall be developed based on national consensus standards such as 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.

The fabricator's quality assurance inspector (QAI) shall inspect to the approved shop drawings the following as a minimum, as applicable:

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

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

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

(4) Tolerances for shop fabrication in accordance with Section 6 of the *Code of Standard Practice* and Chapter NM

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

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

(2) Steel deck and 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 7.13 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 by the owner or the engineer of record or their designee in accordance with Section 4.4 of the *Code of Standard Practice*, prior to fabrication or erection, as applicable:

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

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

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

2. Available Documents for Steel Construction

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

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

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

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

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

(5) For anchor rods and threaded rods, copies of material test reports in accordance with *Specification* Section A3.4.

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

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

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

(9) Welding procedure specifications (WPS).

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

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

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

(13) Fabricator's or 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.

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 6.1.4, and AWS D1.6, clause 6.2, 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 6.14.6, and AWS D1.6, clause 6.2.7, 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 are those inspections performed by qualified personnel to ensure that the work is performed in accordance with the construction documents.

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

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

2. Quality Assurance

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

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 specification, 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 the work being performed.

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

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 are those inspections performed by the QAI to ensure that the work is performed in accordance with the construction documents.

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

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

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 quality control inspection and QAI so that the inspection functions are performed by only one party. Where QA relies upon inspection functions performed by personnel qualified for quality control inspection, the approval of the engineer of record and the AHJ is required, and the procedure must be stated in the QA program.

4. Inspection of Welding

Observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the construction documents. 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.

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.

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 are as follows:

O—The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.

P—These tasks shall be performed for each welded joint or member.

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

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

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

UT shall be performed by qualified NDE personnel on complete-joint-penetration (CJP) groove welds subject to transversely applied tension loading in butt, T- and corner joints, in materials 5⁄16 in. (8 mm) thick or greater.

As a minimum, CJP welds shall be 10% inspected by ultrasonic examination or radiographic examination. The examination shall be either 10% of each weld or 100% of one weld in 10.

As a minimum, 10% of partial-joint-penetration (PJP) welds shall be inspected by magnetic particle examination or liquid penetrant examination. This examination shall be either 10% of each weld or 100% of one weld in 10.

> **User Note:** The fabricator or erector should identify a method of quantifying the population of welds to be tested. This can be done either by part, drawings, WPS, or other means that identifies the size of the weld population from which the 10% sample is selected. 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

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

5d. Ultrasonic and Radiographic Testing Rejection Rate

The ultrasonic and radiographic testing rejection rate shall be determined as the number of welds containing defects divided by the number of welds completed. Welds that contain acceptable discontinuities shall not be considered as having defects when the rejection rate is determined. For evaluating the rejection rate of continuous welds over 3 ft (1 m) in length where the effective throat is 1 in.

(25 mm) or less, each 12-in. (300 mm) increment, or fraction thereof, shall be considered as one weld. For evaluating the rejection rate on continuous welds over 3 ft (1 m) in length where the effective throat is greater than 1 in. (25 mm), each 6 in. (150 mm) increment, or fraction thereof, shall be considered one weld.

5e. Reduction of Rate of Ultrasonic and Radiographic Examination

For projects that contain 40 or fewer welds, there shall be no reduction in the ultrasonic testing rate. The rate of UT or RT is permitted to be reduced if approved by the engineer of record or the AHJ. Where the initial rate of UT or RT is 100%, the NDE rate for an individual welder or welding operator is permitted to be reduced to 25%, provided the rejection rate, the number of welds containing unacceptable defects divided by the number of welds completed, is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds shall be made for such reduced evaluation on each project.

5f. Increase in Rate of Ultrasonic and Radiographic Examination

For structures in which the initial rate for UT or RT is 10%, the NDE rate for an individual welder or welding operator shall be increased to 100% if the rejection rate (the number of welds containing unacceptable defects divided by the number of welds completed) exceeds 5% of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds on each project shall be made prior to implementing such an increase. If the rejection rate for the welder or welding operator falls to 5% or less based on at least 40 completed welds, the rate of UT and RT may be decreased to 10%.

5g. Documentation

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

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 are not applicable. The QAI need not be present during the installation of fasteners in snug-tight joints.

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

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

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

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.

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 on site is common.

8. Other Inspection Tasks

The fabricator's QAI shall inspect the fabricated steel to verify compliance with the details shown on the shop drawings.

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 erection drawings.

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 chapter.

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

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

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

P—Perform these tasks for each steel element.

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) wall 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.

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 Wall 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 steel anchors	P	P
Document acceptance or rejection of steel elements	P	P

TABLE NN6.3
Inspection of SC Wall 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

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.

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 a nonlinear 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 FOR PONDING

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 are 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 design 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 may be used provided they are substantiated or verified by test. The possible increased deflection that may occur due to elevated temperatures shall be considered in the design.

N4.2 STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

3a. Thermal Elongation

Replace section with the following:

The coefficients of expansion shall be taken as follows:

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

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

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

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)	$k_c = f_c'(T)/f_c'$	$E_c(T)/E_c$	$\varepsilon_{cu}(T)$, %
	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 Section NA3.1d are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the provisions of Section NA3.1d. If the notch toughness so determined does not meet the provisions of Section NA3.1d, the 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.

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

3. Serviceability Evaluation

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

N5.4. EVALUATION BY LOAD TESTS

1. Determination of Live Load Rating by Testing

To determine the live load rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the EOR 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 drawings, material test reports, and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the required strength of the structure, including members and connections, is adequate to withstand the load combinations of either Section NB2.5 or NB2.6, whichever is applicable.

APPENDIX N6

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 SECOND-ORDER ANALYSIS

No changes to Appendix 8 of the Specification.

APPENDIX N9

STEEL-PLATE COMPOSITE (SC) WALLS

This appendix addresses the requirements for steel-plate composite (SC) walls in safety-related structures for nuclear facilities. The provisions of this appendix are limited to SC walls consisting of two steel plates (faceplates) composite with structural concrete between them, where the faceplates are anchored to concrete using steel anchors and connected to each other using ties.

The appendix is organized as follows:

N9.1. Design Requirements
N9.2. Analysis Requirements
N9.3. Design of SC Walls
N9.4. Design of SC Wall 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 apply to SC walls:

(a) The SC section thickness, t_{sc}, shall not exceed 60 in. (1500 mm). For exterior SC walls, the minimum t_{sc} shall be 18 in. (450 mm). For interior SC walls, the minimum t_{sc} shall be 12 in. (300 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 0.050, where ρ is determined as follows:

$$\rho = \frac{2t_p}{t_{sc}} \tag{A-N9-1}$$

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 65 ksi (450 MPa).

(e) The specified compressive strength of the concrete, f'_c, shall not be less than 4 ksi (28 MPa) nor more than 8 ksi (55 MPa). Lightweight concrete shall not be used.

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

(g) Composite action shall be provided between faceplates and concrete using steel anchors, 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 effective rupture strength per unit width, $F_u A_{sn}$, shall be greater than the yield strength per unit width, $F_y A_s$,

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)

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

(k) Steel ribs, if applicable, shall be embedded into the concrete no more than the lesser of 6 in. (150 mm) or the embedment depth of the steel anchor minus 2 in. (50 mm). The ribs shall be welded to the faceplates and anchored in the concrete to develop 100% of their nominal yield strength.

(l) Splices between faceplates shall be welded using complete-joint-penetration (CJP) groove welds or bolted to develop the nominal yield strength of the two (spliced) faceplates.

User Note: This appendix was developed for straight SC walls. For curved SC walls, the effects of the curvature on section detailing (faceplate slenderness, tie requirements, etc.) and design strengths (section available strengths, interaction of forces and moments, etc.) need to be evaluated.

2. Design Basis

For design purposes, SC walls 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 walls and their connections shall be determined through an elastic finite element analysis for the applicable load combinations, except as stated in Section N9.1.6c.

User Note: As discussed in Section N9.1.6c, 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 straight SC walls need not be performed if the conditions of ACI 318 or ACI 318M, Section 6.2.5, are satisfied. If the conditions of ACI 318 or ACI 318M, Section 6.2.5, are not satisfied, second-order effects shall be considered.

> **User Note:** Second-order analysis is not warranted in most cases because the typical SC walls in safety-related nuclear facilities tend to be stocky and are braced against sway-related P-Δ effects. Also, their unbraced heights between adjacent floors generally meet the slenderness criteria of ACI 318 or ACI 318M, Equations 6.2.5a, 6.2.5b and 6.2.5c, and therefore, P-δ effects are negligible as well. In rare situations where the requirements of Section 6.2.5(b) are not satisfied, the limitations associated with the first-order analysis method in *Specification* Appendix 7, Section 7.3, are generally met. If the limitations associated with the first-order analysis are not met, second-order effects can be accounted for using *Specification* Appendix 8, when applicable.

3. Faceplate Slenderness Requirement

Faceplates shall be anchored to concrete using steel anchors, ties, or a combination thereof. The width-to-thickness ratio of the faceplates, b/t_p, shall be limited as follows:

$$\frac{b}{t_p} \le 1.0\sqrt{\frac{E_s}{F_y}} \tag{A-N9-2}$$

where

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

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

b = largest unsupported length of the faceplate between rows of steel anchors or ties, in. (mm)

t_p = thickness of faceplate, in. (mm)

4. Requirements for Composite Action

4a. Classification of Steel Anchors

Connectors with interfacial slip of at least 0.20 in. (5 mm), while maintaining a resistance greater than 90% of the peak shear strength, shall be classified as yielding steel anchors. Steel anchors not meeting this requirement shall be classified as nonyielding steel anchors. Steel headed stud anchors shall be classified as yielding steel anchors, and the available shear strength, Q_{cv}, shall be obtained using the *Specification*. Classification and available strength, Q_{cv}, for all other types of steel anchors shall be established through testing.

> **User Note:** Requirements for steel headed stud anchors are provided in *Specification* Sections I8.1 and I8.3.

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

4b. Spacing of Steel Anchors

Steel anchors 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

$$s \le c_1 \sqrt{\frac{Q_{cv} L_d}{T_p}} \tag{A-N9-3}$$

where

L_d = development length, in. (mm)
$\le 3t_{sc}$
Q_{cv} = available shear strength of steel anchor determined in accordance with Section N9.1.4a, kips (N)
T_p = $F_y t_p$ for LRFD, kip/in. (N/mm)
= $F_y t_p/1.5$ for ASD, kip/in. (N/mm)
c_1 = 1.0 for yielding steel anchors
= 0.7 for nonyielding steel anchors

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

$$s \le c_1 \sqrt{\frac{Q_{cv} l}{V_c / 0.9 t_{sc}}} \tag{A-N9-4}$$

where

V_c = available out-of-plane shear strength per unit width of SC panel section, kip/ft (N/m)
l = unit width, 12 in./ft (1000 mm/m)
t_{sc} = SC section thickness, in. (mm)

> **User Note:** Steel anchor 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 steel anchor spacing is designed to achieve interfacial shear strength to be greater than the available out-of-plane shear strength determined in accordance with Section N9.3.5.

5. Tie Requirements

The opposite faceplates of SC walls shall be connected to each other using ties consisting of individual components such as structural shapes, frames or bars. Ties shall have spacing no greater than the section thickness, t_{sc}.

User Note: Ties serve a dual purpose. They provide structural integrity by preventing section splitting and 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

$$F_{ny} \leq 0.8F_{nr} \tag{A-N9-5}$$

where

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

F_{ny} = nominal yield strength of the tie, kips (N)

Otherwise, ties shall be classified as nonyielding shear reinforcement.

User Note: The nominal strength of the associated connection is the nominal strength of the welded or bolted connection of the tie to the faceplate.

5b. Required Tensile Strength for Ties

The required tensile strength, F_{req}, for individual ties is given as

$$F_{req} = \left(\frac{t_p F_y t_{sc}}{4}\right)\left(\frac{s_{tt}}{s_{tl}}\right)\left[\frac{6}{18\left(\frac{t_{sc}}{s_{tl}}\right)^2 + 1}\right] \tag{A-N9-6}$$

where

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

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

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

User Note: A tie may be a single 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 required tensile strength, F_{req}, is for each individual tie.

6. Design for Impactive and Impulsive Loads

6a. Dynamic Increase Factors

Dynamic increase factors (DIF) based on the strain rates involved are permitted to be applied to static material strengths of steel and concrete for purposes of determining section strength, but shall not exceed those specified in Table A-N9.1.1.

The DIF shall be limited to 1.0 for all materials where the dynamic load factor associated with the impactive or impulsive loading is less than 1.2.

User Note: DIF values are from NEI 07-13, *Methodology for Performing Aircraft Impact Assessments for New Plant Designs*, Revision 8P.

6b. Ductility Ratios

The available strength of SC walls for impactive and impulsive loads may be governed by flexural yielding or out-of-plane shear failure. SC walls shall be classified as flexure-controlled if their available strength for the limit state of flexural yielding is less than their available strength for the limit state of out-of-plane shear failure by at least 25%; otherwise, they shall be classified as shear-controlled.

The required ductility ratio, μ_r, of flexure-controlled SC walls shall not exceed 10, where μ_r is given as:

$$\mu_r = \frac{D_m}{D_y} \tag{A-N9-7}$$

where

D_m = maximum displacement from analysis (in accordance with Section N9.1.6c), in. (mm)

D_y = effective yield displacement (displacement associated with the effective yield point as defined under permissible ductility ratio), in. (mm)

For flexure, the effective yield displacement, D_y, shall be established using the cross-sectional effective flexural stiffness for analysis, EI_{eff}, calculated using Equation A-N9-8 or Equation A-N9-8M.

For shear-controlled SC walls with yielding shear reinforcement spaced at section thickness divided by two or smaller, the ductility ratio demand shall not exceed 1.6. For shear-controlled SC walls with other configurations of yielding or nonyielding shear reinforcement, the ductility ratio demand shall not exceed 1.3.

The ductility ratio demand shall not exceed 1.3 for axial compressive impactive or impulsive loads.

TABLE A-N9.1.1
Dynamic Increase Factors (DIF)

Material	DIF	
	Yield Strength	Ultimate Strength
Carbon steel plate	1.29	1.10
Stainless steel plate	1.18	1.00
Reinforcing steel Grade 40 Grade 60	 1.20 1.10	 1.05 1.05
Concrete compressive strength	—	1.25
Concrete shear strength	—	1.10

6c. Response Determination

The response of SC walls subjected to impulsive loads shall be determined by one of the following methods:

(a) The dynamic effects of the impulsive loads are considered by calculating a dynamic load factor (DLF). The resistance available for the impulsive load is at least equal to the peak of the impulsive load transient multiplied by the DLF, where the calculation of the DLF is based on the dynamic characteristics of the structure and impulsive load transient.

(b) The dynamic effects of impulsive loads are considered by using impulse, momentum, and energy balance techniques. Strain energy capacity is limited by the ductility criteria in Section N9.1.6b.

(c) The dynamic effects of impulsive loads shall be considered by performing a time-history dynamic analysis. The mass and inertial properties shall be included, as well as the nonlinear stiffnesses of the structural members under consideration. Simplified bilinear definitions of stiffness are permitted. The maximum predicted response is governed by the ductility criteria in Section N9.1.6b.

User Note: Rational methods to consider dynamic effects of impulsive loads are discussed in the Commentary.

Design for impactive loads shall satisfy the criteria for both local effects and overall structural response. Local impact effects shall include perforation of the SC wall.

The faceplate thickness required to prevent perforation shall be at least 25% greater than that calculated using rational methods.

User Note: One rational method for calculating the faceplate thickness to prevent perforation is provided in the Commentary.

7. Design and Detailing around Openings

7a. Design and Detailing Requirements around Small Openings

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

(a) Design and detailing for a free edge at the opening perimeter

(1) Analysis is permitted to be performed without modeling the opening.

(2) The panel section where the opening is located shall be evaluated considering 25% reduction in all available strengths.

(3) Reentrant corners of noncircular or nonoval 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 no greater than one-quarter of the SC section thickness, t_{sc}.

(b) Design and detailing for a fully developed edge at the opening perimeter

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

(1) Reentrant corners of noncircular or nonoval 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.

(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) A welded flange, made from plate material with nominal yield strength equal to or greater than that of the surrounding faceplate, shall be fitted at each end of the sleeve. The flange shall be at least as thick as the faceplate, and it shall extend a distance of at least the section thickness beyond the opening perimeter. The flange shall be connected to the sleeve using CJP groove welds.

(5) The flange shall be joined with the surrounding faceplate in one of the following ways:

(i) If the flange is less than 25% thicker than the surrounding faceplate, the faceplate shall be joined with the sleeve using a CJP groove weld and the flange shall be joined with the faceplate using the maximum size fillet weld permitted by the *Specification*; or

(ii) If the flange is greater than or equal to 25% thicker than the faceplate, the faceplate shall be joined with the flange only along its outer perimeter with a CJP groove weld.

7b. Design and Detailing Requirements around 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 wall. Design and detailing shall be accomplished as follows:

(a) Design and detailing for a free edge at the opening perimeter

(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 nonoval 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 no greater than one-quarter of the SC section thickness, t_{sc}.

(b) Design and detailing for a fully developed edge at the opening perimeter

Fully developed SC walls around large openings shall be modeled and designed considering the physical boundary of the opening and shall follow the provisions for fully developed 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 need to be detailed in accordance with Section N9.1.7b by taking into account the nature of boundary conditions provided around the opening.

7c. Bank of Small Openings

The region affected by a concentrated bank of small openings shall be considered as a large opening when the clear distance between adjacent small openings is equal to or less than

(a) $2t_{sc}$ for openings designed and detailed for the free edge at the opening perimeter

(b) t_{sc} for openings designed and detailed for the fully developed edge at the opening perimeter

The physical dimensions of the large opening shall be equal to the distance between the outermost edges of the bank of small openings.

User Note: Dimensions of the as-modeled large opening are as discussed in Section N9.1.7b.

N9.2. ANALYSIS REQUIREMENTS

1. General Provisions

The following provisions apply to the analysis of SC walls.

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

User Note: Guidelines 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 walls.

2. Effective Stiffness for Analysis

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

$$EI_{eff} = \left(E_s I_s + c_2 E_c I_c\right)\left(1 - \frac{\Delta T_{savg}}{150}\right) \geq E_s I_s \qquad \text{(A-N9-8)}$$

$$EI_{eff} = \left(E_s I_s + c_2 E_c I_c\right)\left(1 - \frac{\Delta T_{savg}}{83}\right) \geq E_s I_s \qquad \text{(A-N9-8M)}$$

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 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)
= calibration constant for determining effective flexural stiffness

c_2 = $0.48\rho' + 0.10$

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

l = unit width, 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
= $2t_p/t_{sc}$
$\bar{\rho}$ = 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 $S_{ryx} \le S_{cr}$

$$GA_{eff} = GA_{uncr} = GA_s + G_cA_c \tag{A-N9-9}$$

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 = shear modulus of elasticity of steel
= 11,200 ksi (77 200 MPa) for carbon steel
= 10,800 ksi (74 500 MPa) for stainless steel
G_c = shear modulus of concrete
= $772\sqrt{f_c'}$, ksi ($2000\sqrt{f_c'}$, MPa)
S_{rxy} = required membrane in-plane shear strength per unit width in the panel section, kip/ft (N/m)

$$S_{cr} = \frac{0.063\sqrt{f_c'}}{G_c}GA_{uncr} \tag{A-N9-10}$$

$$= \frac{0.17\sqrt{f_c'}}{G_c}GA_{uncr} \tag{A-N9-10M}$$

GA_{uncr} = $GA_s + G_cA_c$
f_c' = specified compressive strength of concrete, ksi (MPa)
l = unit width, 12 in./ft (1000 mm/m)

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

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

where

$$GA_{cr} = 0.5\bar{\rho}^{-0.42} GA_s \quad \text{(A-N9-12)}$$

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

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

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

(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 wall 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 walls shall be modeled in the elastic finite element analyses as follows:

(a) Poisson's ratio, ν_m, thermal expansion coefficient, α_m, and thermal conductivity, k_m, shall be taken as that of the concrete.

(b) Model section thickness, t_m, and the material elastic modulus, 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) Material density, 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) Specific heat, 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 Accident Thermal Conditions

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 wall interior regions caused by the thermal gradients shall not exceed $M_{r\text{-}th}$, where

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

where

EI_{eff} = effective flexural stiffness for analysis of SC walls per unit width, kip-in.^{2}/ft (N-mm^2/m)
α_s = thermal expansion coefficient of faceplate in °F^{-1} (°C^{-1})
ΔT_{sg} = maximum temperature difference between faceplates due to accident thermal conditions in ºF (ºC)

User Note: Analysis for thermal loads may predict moments higher than $M_{r\text{-}th}$ defined above because it does not directly account for the self-limiting effect due to concrete cracking. 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. 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 demand type shall be calculated by averaging the demand 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 demand over panel sections no larger than the section thickness in length and width.

The required strengths for the panel sections of SC walls for each demand type 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 y, kip/ft (N/m)

V_{ry} = required out-of-plane shear strength per unit width along edge parallel to direction x, kip/ft (N/m)

x, y = local coordinate axes in the plane of the wall associated with the finite element model

N9.3 DESIGN OF SC WALLS

The tensile strength contribution of concrete infill and the contribution of steel ribs to the available strengths of SC walls shall be ignored.

1. Uniaxial Tensile Strength

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

For the following variables, the definitions replace those in Specification Section I2.1b:

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

$$= F_y A_{sn} + 0.85 f_c' A_c \tag{A-N9-16}$$

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

$$= \pi^2 EI_{eff} / L^2 \tag{A-N9-17}$$

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)

$$= E_s I_s + 0.60\, E_c I_c \tag{A-N9-18}$$

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

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

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

l = unit width, 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 wall panel sections shall be determined for the limit state of yielding as follows:

$$M_n = F_y\left(A_s^F\right)\left(0.9t_{sc}\right) \tag{A-N9-19}$$

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

where

A_s^F = gross cross-sectional 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)

User Note: The nominal flexural strength per unit width, M_n, can also be calculated using reinforced concrete principles (refer to ACI 349 or ACI 349M, Section 10.2).

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:

$$V_{ni} = \kappa F_y A_s \tag{A-N9-20}$$

$$\phi_{vi} = 0.90 \text{ (LRFD)} \qquad \Omega_{vi} = 1.67 \text{ (ASD)}$$

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)

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

κ = $1.11 - 5.16\bar{\rho} \le 1.0$

$\bar{\rho}$ = strength-adjusted reinforcement ratio, determined using Equation A-N9-13 or Equation A-N9-13M

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:

$$\phi_{vo} = 0.75 \text{ (LRFD)} \qquad \Omega_{vo} = 2.00 \text{ (ASD)}$$

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 for SC panel sections 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

$$V_{conc} = 0.05\left(f_c'\right)^{0.5} t_c l \tag{A-N9-22}$$

$$= 0.13\left(f_c'\right)^{0.5} t_c l \tag{A-N9-22M}$$

$$V_s = \xi p_s F_t \left(l/s_{tt}\right) \le 0.25\left(f_c'\right)^{0.5} t_c l \tag{A-N9-23}$$

$$= \xi p_s F_t \left(l/s_{tt}\right) \le 0.67\left(f_c'\right)^{0.5} t_c l \tag{A-N9-23M}$$

F_t = nominal tensile strength of ties, kips (N)
l = unit width, 12 in./ft (1000 mm/m)
p_s = t_c/s_{tl}
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 upper limit on V_s is based on ACI 349 or ACI 349M, Section 11.5.7.9, which limits the maximum possible contribution of shear reinforcement to out-of-plane shear strength to $0.25\left(f_c'\right)^{0.5} A_c$ or $0.67\left(f_c'\right)^{0.5} A_c$ (S.I.), where A_c is the area of concrete per unit width.

(b) The nominal out-of-plane shear strength per unit width for SC panels 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 or Equation A-N9-23M, taking both ξ and p_s as 1.0.

6. Strength Under Combined Forces

6a. Out-of-Plane Shear Forces

The interaction of out-of-plane shear forces shall be limited by Equation A-N9-24.

(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:

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

where

Q_{cv}^{avg} = weighted average of the available interfacial shear strengths of ties and steel anchors while accounting for their respective tributary areas and numbers, 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 = unit width, 12 in./ft (1000 mm/m)

s = spacing of steel anchors, 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 reinforcement and yielding steel anchors
= 0.5 for panel sections with either nonyielding shear reinforcement or nonyielding steel anchors

For design in accordance with *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 in accordance with *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}/Ω_{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 Equation A-N9-24.

User Note: The interaction equation is based on ACI 349 or ACI 349M, Appendix D, Commentary RD.7, which is applicable to connectors with ductile and nonductile limit states. The second bracketed term in the interaction equation uses the vector sum of the shears V_{rx} and V_{ry}, and is obtained by manipulation of Equation A-N9-4.

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. They 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 $S_{r,max} + S_{r,min} \geq 0$

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

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

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

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

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

where

$$S_{r,max}, S_{r,min} = \frac{S'_{rx}+S'_{ry}}{2} \pm \sqrt{\left(\frac{S'_{rx}-S'_{ry}}{2}\right)^2 + \left(S'_{rxy}\right)^2} \tag{A-N9-28}$$

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

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

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

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 $S_{rx} > -0.6P_{no}$ and $j_x = 0.67$ if $S_{rx} \le -0.6P_{no}$

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 $S_{ry} > -0.6P_{no}$ and $j_y = 0.67$ if $S_{ry} \le -0.6P_{no}$

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)

Alternately, 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 $S'_{rx} + S'_{ry} \ge 0$

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

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

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

(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

α = V_{ci} / T_{ci}

β = V_{ci} / P_{ci}

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 in accordance with *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), where T_{ni} is calculated using the nominal tensile strength in accordance with Section N9.3.1

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 in accordance with *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), where T_{ni} is calculated using the nominal tensile strength in accordance with Section N9.3.1

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

7. Strength of Composite Linear Members in Combination with SC Walls

Linear composite members are permitted to be used in conjunction with SC walls. They shall be designed in accordance with *Specification* Chapter I.

N9.4 DESIGN OF SC WALL CONNECTIONS

This section addresses design requirements for (a) splices between SC wall sections, (b) splices between SC wall and reinforced-concrete (RC) wall sections, (c) connections at the intersections of SC walls, (d) connections at the intersection of SC with RC walls, (e) anchorage of SC walls to RC basemats, and (f) connections of SC walls to RC slabs.

1. General Provisions

Wall-to-wall, wall anchorage, and wall 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, rebar mechanical couplers, and direct bearing in compression. Force transfer mechanisms involving connectors of the same type (from the list above, e.g., anchor rods) 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 connection design examples, refer to AISC Design Guide 32, *Design of Modular Steel-Plate Composite Walls for Safety-Related Nuclear Facilities*.

2. 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 capacity 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.

3. 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 is determined in accordance with *Specification* Section I8.3.

(b) For welds and bolts, the available strength is determined in accordance with *Specification* Chapter J.

(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.

COMMENTARY

on the Specification for Safety-Related Steel Structures for Nuclear Facilities

June 28, 2018

INTRODUCTION

The *Specification for Safety-Related Steel Structures for Nuclear Facilities* is intended to be complete for normal design usage in the design, fabrication and erection of safety-related steel structures for nuclear facilities in conjunction with the AISC *Specification for Structural Steel Buildings* and Commentary (ANSI/AISC 360-16).

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

The Nuclear Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.

CHAPTER NA
GENERAL PROVISIONS

Modify Chapter A of the Specification Commentary as follows:

NA1. SCOPE

Replace section with the following:

The *Specification for Safety-Related Steel Structures in Nuclear Facilities*, hereafter referred to as the Nuclear Specification, follows the lead of the 2016 AISC *Specification for Structural Steel Buildings* (AISC, 2016a), hereafter referred to as the *Specification*, and modifies the provisions of previous AISC Nuclear Specifications to make it compatible with the *Specification*.

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

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

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

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

The Nuclear Specification may be applicable to all structural steel members in nuclear facilities. Specifically excluded from the Nuclear Specification are the pressure retaining components, for example, pressure vessels, valves, pumps and piping. For the materials, design, fabrication and examination of plate and shell component supports, readers are directed to the requirements of Subsection NF of Section III of the ASME *Boiler and Pressure Vessel Code* (ASME, 2015a).

The 2016 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016b), hereafter referred to as the *Seismic Provisions*, is intended for the design and construction of steel members and connections in the seismic force-resisting systems in buildings for which the required strengths resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response.

The requirements of *Seismic Provisions* Sections A3, D1 and D2 are recommended for members and connections subject to localized inelastic response due to the action of certain load actions (such as impact loads, for which local inelastic response is considered acceptable). Conformance with the cited *Seismic Provisions* sections will help the affected members and connections to withstand the load effects without overcoming their force or deformation capacities, as applicable.

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

The 2016 AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2016c), hereafter referred to as the *Code of Standard Practice*, defines the practices that are the commonly accepted standards of custom and usage for structural steel fabrication and erection. As such, the *Code of Standard Practice* is primarily intended to serve as a contractual document to be incorporated into the contract between the buyer and seller of fabricated structural steel. Some parts of the *Code of Standard Practice*, however, form the basis for some of the provisions in the Nuclear Specification. Therefore, the *Code of Standard Practice* is referenced in selected locations in the Nuclear Specification to maintain the ties between those documents, where appropriate.

NA3. MATERIAL

Modify this section as follows:

1. Structural Steel Materials

Add the following:

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

1a. ASTM Designations

Add the following:

Plates. Plate materials ASTM A537/A537M and ASTM A738/A738M are permitted based on ASME *Boiler and Pressure Vessel Code*, Section III, Division 2, Sub-Article CC-2510. These materials may be used for steel-plate composite (SC) wall construction. ASME *Boiler and Pressure Vessel Code*, Section III, Division 2, Sub-Article CC-2510, refers to Table D2-I-2.2, "Material for Containment Liners," which contains a multitude of materials, including pipe, forgings, etc., that are not applicable to SC wall construction. Of the materials mentioned under the "Plate" heading, the materials satisfying the requirements of Appendix N9 have been listed in the Nuclear Specification.

Bars. The unmodified martensitic grade of ASTM A276/A276M is not readily weldable. Martensitic steels are susceptible to excessive hardening with consequent risk of cracking during welding.

1c. Rolled Heavy Shapes

Modify this section as follows:

1c. Rolled Heavy Shapes

and

1d. Built-Up Heavy Shapes

Add the following:

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

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

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

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

2. Steel Castings and Forgings

Delete the following:

Design and fabrication of cast and forged steel components are not covered in the Nuclear Specification.

5. Consumables for Welding

Add the following:

Because nuclear facilities sometimes utilize stainless steel structural materials, AWS D1.6/D1.6M (AWS, 2007), AWS A5.4/A5.4M (AWS, 2014), and AWS A5.9/A5.9M (AWS, 2012) have been added to the Nuclear Specification. Previous AISC nuclear specifications referenced ASME *Boiler and Pressure Vessel Code,* Section IX, for stainless welding, but with the availability of AWS D1.6/D1.6M, the reference to Section IX was deleted from the 2012 Nuclear Specification.

NA4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

Add the following:

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

Add the following section:

NA5. QUALITY ASSURANCE

This section has been added to comply with the requirements of the authority having jurisdiction (AHJ). For design of safety-related structures, this provision has been clarified to require the designer to follow the latest code, ASME NQA-1 (ASME, 2015b), or other approved standards; these other approved standards would include ANSI N45.2 (ANSI, 1977) documents, which pertain to older nuclear plants.

CHAPTER NB
DESIGN REQUIREMENTS

Modify Chapter B of the Specification Commentary as follows:

NB2. LOADS AND LOAD COMBINATIONS

Replace section with the following:

Inclusion of *F* and *H* loads is required, because unlike linear elements (beams, columns, braces, etc.) of steel buildings, plate or shell-type structures of safety-related nuclear facilities may be subjected to soil and fluid pressures. The pertinent load combinations come from ACI 349 or ACI 349M (ACI, 2013), SRP 3.8.3 and 3.8.4 (NRC, 2013a, b), and RG 1.142 (NRC, 2001).

1. Normal Loads

Dead and live loads form a generic category of normal loads. During initial design, the values of most of the piping loads and suspended system loads (HVAC, cable trays, etc.) are not available, and the load allowance for these items is included in *L* as an area-averaged load. Once the final attachment loads are determined, the initial load assumptions should be confirmed. When designing for weights or pressures from fluids, either existing in the building or due to hydrostatic heads, both cases (with fluid present or absent) should be evaluated in order to establish the governing load condition. When a detailed dynamic analysis is performed for crane systems, elevators, or other moving machinery, the resulting load with dynamic amplification may be used in lieu of the load increases (dynamic impact factors) specified in ASCE/SEI 7-16 (ASCE, 2016), or similar documents.

The weight of the crane trolley and bridge does not include the lifting load. The lifting load is part of load *C* in the load combination. Unlike other types of dead loads, the crane trolley and bridge can have many positions during the operation of the plant. The gravity structural analysis of the building must consider all the trolley and bridge positions that produce the highest responses in the building structural components.

Sections NB2.1 and NB2.2 state that the snow load, *S*, and wind load, *W*, are as stipulated in ASCE/SEI 7-16 for Risk Category IV facilities. Risk Category IV facilities are defined in Table 1.5-1 of ASCE/SEI 7-16 as those for which continued function following the occurrence of a natural phenomenon hazard is essential for public health and safety. For such facilities, ASCE/SEI 7-16 requires that the nominal load otherwise determined for ordinary buildings and other structures be increased by an importance factor. This importance factor is 1.2 for snow load. These increases are tantamount to requiring Risk Category IV facilities to be designed for 100-year mean recurrence interval snow events. The importance factor for wind loads has been deleted (from previous editions of ASCE/SEI 7) due to changes in new wind hazard maps.

4. Abnormal Loads

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

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

(b) A break in a small line containing high-temperature fluids or steam. This would result in a long-term high temperature and associated pressure loading.

(c) Other extreme load phenomena that have a probability of occurrence larger than 10^{-7} events per year, the consequence of which could lead to release of radiation in excess of 10 CFR Part 100 limits (Code of Federal Regulations, 2007).

5. Load and Resistance Factor Design (LRFD)

The Nuclear Specification permits design for strength by either the load and resistance factor design (LRFD) method or the allowable strength design (ASD) method.

The load combinations stem from a probability-based study of load combinations for design of nuclear power plants (Hwang et al., 1987). The probabilistic methodology in that study is consistent with that used to develop the probability-based load combination requirements appearing in ASCE/SEI 7-16 (ASCE, 2016), Galambos et al. (1982), and Ellingwood et al. (1982). The load statistics for operating and abnormal plant conditions were obtained from a consensus estimation survey of operating load in nuclear facilities (Hwang et al., 1983).

Load Combination NB2-4 for severe environmental loads includes the wind load, W, from Section 26 of ASCE/SEI 7-16 (ASCE, 2016). This wind load addresses extreme nontornadic wind effects from extratropical storms and hurricanes. Tornadic wind effects are defined by W_t, and are addressed in Load Combination NB2-7 for extreme environmental effects. The extreme environmental loads, W_t and E_s, as specified in NUREG-0800 (NRC, 2007b) and in 10 CFR Part 50 (Code of Federal Regulations, 2010), are design-basis events and thus appear in the load combinations with load factors of unity.

Dynamic load effects should be considered with maximum values assumed acting simultaneously, unless actual time history analysis shows a different time-phase relationship, in which case, loads may be combined as a function of time. Loads due to postulated accidents and natural phenomena often yield dynamic response of short duration and rapidly varying amplitude in the exposed structures and components. For some loading phenomena, accident analysis provides a definitive time history response and allows a straightforward addition of responses where more than one load is acting concurrently. In other cases, no specified time-phase relationship exists, either because the loads are random in nature or because the loads have simply been postulated to occur together (for example, loss of coolant accident and safe shutdown earthquake) without a known or defined coupling. Where a defined time-phase relationship is lacking, system designers have utilized several approaches to account for the potential interaction of the loads. One approach, the so-called absolute or linear summation (ABS) method, linearly adds the absolute values of the peak structural response due to the individual dynamic loads. A second approach, referred to as the square root of the sum of the squares (SRSS) method, yields a combined response equal to the square root of the sum of the squares of the peak responses due to the individual dynamic loads. Research conducted over the past two decades shows that this method of combining dynamic responses is conservative unless the structural responses are stochastically dependent. The SRSS method of load combination is acceptable to the U.S. Nuclear Regulatory Commission (NRC, 1980), contingent upon the performance of a linear elastic dynamic analysis. Thus, the loads from a loss-of-coolant accident (LOCA) and a seismic event combined in Load Combination NB2-9 may be combined by the SRSS method, provided that the responses are determined by elastic analysis. However, this does not prohibit the use of more conservative load combination schemes. In all cases, resultant dynamic loads shall be combined absolutely, considering both maximum positive and negative values, with applicable static loads.

Any portion of thermal deformations that is restrained (because of the structure's external and internal restraints) leads to forces and/or moments in the restrained members. Accordingly, realistic modeling of the member stiffness, as well as the stiffness of external restraints and connections, is recommended for estimating the magnitudes of thermally induced member forces and moments, which would otherwise be overestimated if the external restraints and connections are assumed to be rigid. In this regard, the prescriptive rules for implementation of direct analysis in accordance with *Specification* Chapter C, which already includes reduction of member axial and flexural stiffness, are appropriate and beneficial for reducing the magnitudes of thermally induced member forces and moments. Additionally, when applicable, temperature-dependent reduction of steel modulus of elasticity (in accordance with *Specification* Table A-4.2.1) is also appropriate and beneficial for reducing the magnitudes of thermally induced member forces and moments. A rigorous second-order analysis that accounts for large-displacement theory, especially accounting for catenary behavior (when the member end connections are designed to support such behavior), is recommended. The reduction in thermally induced forces due to large-deformation effects and catenary action has been demonstrated by Usmani et al. (2001) and Wang and Yin (2005). The forces and moments associated with the final equilibrium state obtained from rigorous second-order analysis should be used for code checks.

6. Allowable Strength Design (ASD)

The starting point for the development of load combinations for allowable strength design was the load combinations that appear in the 2006 Nuclear Specification (AISC, 2006). These load combinations and accompanying stress limit coefficients were re-examined in the light of recent advances in the *Specification*, as well as the principal action-companion action load combination format followed in ASCE/SEI 7-16 (ASCE, 2016) and in Section NB2.5 of the Nuclear Specification. The allowable strength design load combinations and other considerations in Section NB2.6 stem from this re-examination.

Refer to Commentary Section NB2.5 for additional discussion on the treatment of thermally induced member forces and moments.

NB3. DESIGN BASIS

1. Design for Strength Using Load and Resistance Factor Design (LRFD)

***Add the following paragraph*:**

The strength of steel decreases at elevated temperatures. Where the structural component or system is exposed to sustained temperature in excess of 250°F (121°C), the decrease should be taken into account in determining the design strength. Design values for steel strength at elevated temperature may be obtained from ASME Code Section II, Part D (ASME, 2015a).

2. Design for Strength Using Allowable Strength Design (ASD)

***Add the following paragraph*:**

The strength of steel decreases at elevated temperatures. Where the structural component or system is exposed to sustained temperatures in excess of 250°F (121°C), the decrease should be taken into account in determining the allowable strength. Design values for steel strength at elevated temperatures may be obtained from ASME Code Section II, Part D (ASME, 2015a).

3. Required Strength

***Add the following paragraph*:**

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

8. Design for Serviceability

Add the following:

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

Add the following section:

14. Design Based on Ductility and Local Effects

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

Section NB3.14 permits the load effects from impact or impulsive forces to be determined by inelastic analysis. If an idealized bilinear elastic-plastic stress-strain curve (or load-deflection curve, as appropriate) is used, then the dynamic load effect is determined by modeling the target as a single-degree-of-freedom (SDOF) elastic-plastic system. Note that the effective yield point for the idealized stress-strain (or load-deflection) curve is described in the Glossary definition for permissible ductility ratio and is illustrated in Figure F.3.1 of ACI 349-13 and ACI 349M-13 (ACI, 2013). The required ductility ratio for the equivalent SDOF system is either determined by a nonlinear time-history analysis or, for well-defined impulse functions (rectangular and triangular pulses), selected from established response charts, such as those in Biggs (1964). Alternatively, in case of impactive loads, the required ductility ratio can be conservatively determined using the energy balance method by conservatively assuming that the missile kinetic energy is entirely absorbed by the member by undergoing inelastic deformation. The member adequacy for the resulting inelastic deformation is determined by comparing the calculated member required ductility ratio with the applicable permissible ductility ratio provided in Table NB3.1. Since the design is to be based on the member's own ductility (rather than that of its connections), use of Table NB3.1 requires that the connection strength be 1.30 times that of the member's nominal strength. The 1.30 factor is based on R_y values in Table A3.1 of the AISC *Seismic Provisions*, with the understanding that materials with high yield strength variability (e.g., ASTM A36 and ASTM A53) will not be used for applications involving impulsive and impactive loads.

As an alternative to inelastic design based on use of Table NB3.1, the inelastic analysis can be based on use of refined inelastic analysis that uses a more rigorously determined stress-strain (or load-deflection) curve. Under this approach, the member's adequacy is verified by ensuring that the calculated maximum strain (or deflection) is less than half of the value corresponding to the onset of plastic instability (as seen from the applicable stress-strain or load-deflection curve). Peer review is recommended when exercising the use of the alternative technique because it requires sophisticated inelastic analysis as well as rigorous knowledge of the member behavior.

Regardless of the analysis/design verification method, width-to-thickness ratios for members resisting impulsive/impactive loads through flexure or compression shall conform to the limits in Table NB3.2.

The permissible ductility ratios in Table NB3.1 are based on the following considerations:

(a) Axial Tension: Steel members under axial tension exhibit a ductility equivalent to full strain at ultimate stress. In developing the permitted ductility ratio, the strain at ultimate stress has been assumed to equal one-half the minimum specified percentage elongation at fracture, a safety factor of 2 has been applied to that limit, and the maximum permitted strain has been limited to 0.10.

(b) Flexure: The permissible ductility ratio of 20 for closed sections is based on tests reported in Howland and Newmark (1953). For open sections, the permissible ductility ratio is reduced to 10 when flexure governs and 5 when shear governs. In order to achieve these ductility factors, local buckling and lateral buckling must be prevented by limiting width-to-thickness ratios and unbraced lengths of compression members. For steel plates subject to flexure, the permissible ductility ratio of 20 (same as that for closed section beams) has been conservatively adopted even though plates have larger curvature and rotational ductility (and reserve capacity because of membrane action).

(c) Axial Compression: The strength of short (F_y/F_e < 0.0225) rolled or welded built-up columns is controlled by yielding rather than by buckling, and the permissible ductility ratio is 10. Also, in no case should the permissible ductility ratio be allowed to exceed $\varepsilon_{st}/\varepsilon_y$. As the slenderness increases, buckling controls. Research (Norris et al., 1959) has indicated that for $F_y/F_e > 0.221$, the ductility factor should not be taken to be greater than unity. Between the upper bound $\mu = 10$ when $F_y/F_e = 0.0225$ and lower bound $\mu = 1$ when $F_y/F_e = 0.225$, the permissible ductility ratio is permitted to vary inversely with F_y/F_e.

At the rates of strain that are characteristic of certain impactive or impulsive loads, structural steels exhibit elevated yield strengths, while the strain at the onset of strain hardening and the tensile strength increase slightly. The modulus of elasticity remains nearly constant. Section NB3.14 permits an upward adjustment in the yield stress used to compute nominal strength, R_n, for strain rate effects. Such increases are permitted in other standards. ACI 349 and ACI 349M, Appendix F (ACI, 2013), recommends dynamic increase factors (DIF) of 1.20 for Grade 40 reinforcement and 1.10 for Grade 60 reinforcement. Similar DIF are recommended in *Structural Analysis and Design of Nuclear Plant Facilities* (ASCE, 1986) and in the U.S. NRC Standard Review Plan 3.6.2 (NRC, 2007b). Section NB3.14 permits a 10% increase over the specified minimum yield strength, in the absence of supporting experimental data.

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

Table NB3.1 does not presently address composite members. Composite members are not commonly used in nuclear structures, especially for those members resisting impactive or impulsive loads.

CHAPTER NC
DESIGN FOR STABILITY

Modify Chapter C of the Specification Commentary as follows:

Add the following new paragraph to Section C1:

In considering the effects of elevated temperature, for either the direct analysis method or the effective length method, an elastic analysis is to be performed using the material strength and stiffness properties from *Specification* Appendix 4, Table A-4.2.1.

CHAPTER NI

DESIGN OF COMPOSITE MEMBERS

Modify Chapter I of the Specification Commentary as follows.

Add the following:

The concrete structures in nuclear facilities are designed and constructed using ACI 349-13 or ACI 349M-13 (ACI, 2013). Hence, the applicable requirements of ACI 349-13 or ACI 349M-13, instead of ACI 318-14 or ACI 318M-14 (ACI, 2014), have been included.

CHAPTER NJ

DESIGN OF CONNECTIONS

Modify Chapter J of the Specification Commentary as follows:

NJ2. WELDS

6. Filler Metal Requirements

Add the following:

Additional notch toughness requirements have been incorporated. The provisions are based on the *Seismic Provisions*.

NJ3. BOLTS AND THREADED PARTS

Add the following:

13. Connections for Members Subject to Impactive or Impulsive Loads

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

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

CHAPTER NL

DESIGN FOR SERVICEABILITY

Modify Chapter L of the Specification Commentary as follows:

NL1. GENERAL PROVISIONS

Replace section with the following:

The General Provisions for serviceability for a nuclear plant structure differ from those in the *Specification.* For nuclear plant structures, the focus on serviceability is on the ability of safety-related structures to perform under their intended design conditions that are described in various licensing documents. Deflection and vibration are a primary concern for safety-related structures due to the ramifications that these deflections and vibrations may have on adjacent safety-related systems and components. Due to the robustness of nuclear plant structures, the comfort of the occupants is generally not an issue; accordingly the *Specification* Commentary referral to ASCE/SEI 7 (ASCE, 2016) is not applicable.

CHAPTER NM

FABRICATON AND ERECTION

Modify Chapter M of the Specification Commentary as follows:

NM2. FABRICATION

4. Welded Construction

Add the following:

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

The provisions of ASME *Boiler and Vessel Code*, Section III (ASME, 2015a), are applicable at the weld interface of SC wall elements to elements governed by the ASME Code Section III, Class MC. The applicability of the ASME code is in the context of fabrication requirements.

7. Dimensional Tolerances

Add the following:

Steel-plate composite (SC) construction consists of different phases. Dimensional tolerances are applicable to:

(a) SC wall panels and sub-modules fabricated in the shop and inspected before release

(b) Adjacent SC walls panels, sub-modules, and modules just before connecting them

(c) Erected SC wall modules before concrete casting

(d) Constructed SC structures after concrete casting

SC wall panels are typically fabricated in the shop and then shipped to the field. The overall dimensions of the fabricated SC wall panels are limited by the applicable shipping restrictions. SC wall panels that are shipped by road are limited to 8 to 10 ft (2.4 to 3.0 m) in width and 40 to 50 ft (12 to 15 m) in length, maximum. Additionally, SC wall sub-modules that may consist of corner, joint or splicing modules may also be fabricated in the shop and then shipped to the field. They are subjected to the same size restrictions as the wall panels.

SC wall panels and sub-modules are connected together by welding or bolting to make larger modules. The size and shape of a module is driven by rigging, handling, and field erection/connection considerations. These modules are erected and connected to other modules by welding or bolting to make SC structures. The tolerances given ensure that empty modules are acceptable for construction. The assembled and erected SC modules and structures are filled with concrete.

If the tolerances mentioned in this section are met, no additional considerations in analysis need to be made. Deviations in excess of specified tolerances are not acceptable and need to be given due consideration by performing reconciliatory analysis or by fixing the modules to meet the tolerances. The dimensional tolerances for SC wall panels and sub-modules fabricated in the shop have to be inspected before release for shipping to the site. The dimensional tolerances are primarily for the fabricated panel thickness, t_{sc}, where the tolerance at tie locations is equal to $t_{sc}/200$ rounded up to the nearest 1⁄16 in. (2 mm) and the tolerance in between tie locations is equal to $t_{sc}/100$ rounded up to the nearest 1⁄16 in. (2 mm).

Table C-NM2.1 shows the calculated tolerances for SC wall panels with thickness from 24 to 60 in. (610 to 1500 mm). Due to restricted access within the expanse of the fabricated panels, inspection is required only along the free edges. Shipping restrictions limit the maximum width to 10 ft (3 m). Project specific inspection plans can be developed by the fabricator as needed.

The dimensional tolerance on tie locations is based on the tolerance for shear stud locations in AWS D1.1/D1.1M (AWS, 2015) or AWS D1.6/D1.6M (AWS, 2007), as applicable. This dimensional tolerance also constrains the tolerances for tie spacing and the tie angle with respect to the attached faceplates.

The fabricated panels and sub-modules are shipped to the site and then connected together by welding or bolting to make larger modules. The dimensional tolerance for faceplates of adjoining panels, sub-modules or modules that are connected together by welding is governed by the applicable weld tolerances from the AWS code (AWS D1.1 for carbon steel and AWS D1.6 for stainless steel). For welds that are qualified using project-specific qualification criteria in AWS, the dimensional tolerances should be based on that specified in the qualified weld procedure for the project. No additional squareness or skewed alignment tolerances are needed except those specified for the faceplates of adjoining panels, sub-modules or modules.

The dimensional tolerances for the erected SC modules before concrete placement are based on those for steel structures in the *Code of Standard Practice*. The dimensional tolerances for the constructed SC modules and structures after concrete placement are based on those for concrete construction in ACI 349-13 or ACI 349M-13 (ACI, 2013) and ACI 117-10 or ACI 117M-10 (ACI, 2010). The faceplate waviness requirement following concrete placement is specified to limit excessive faceplate displacement due to concrete placement. Figure C-NM2.1 illustrates how faceplate waviness is measured. The faceplate waviness discussed refers to the total out-of-straightness of the faceplates and is not the net difference between waviness before and after concrete hardening. Corrective measures or reconciliatory analysis need to be performed in case the faceplate waviness requirement is not met.

Benchmarked finite element models (Zhang et al., 2014) were used to study the effect of faceplate waviness on the compressive strength of SC walls with nonslender and slender faceplates. Finite element models of nonslender SC walls with faceplate

Table C-NM2.1
Thickness Tolerances for Fabricated SC Wall Panels and Sub-Modules

Wall Thickness, t_{sc}	**Wall Thickness Tolerance at Tie Locations**	**Wall Thickness Tolerance Between Tie Locations**
in. (mm)	**in. (mm)**	**in. (mm)**
24 (610)	±1⁄8 (±3)	±1⁄4 (±6)
30 (760)	±3⁄16 (±5)	±5⁄16 (±8)
36 (910)	±3⁄16 (±5)	±3⁄8 (±10)
42 (1100)	±1⁄4 (±6)	±7⁄16 (±11)
48 (1200)	±1⁄4 (±6)	±1⁄2 (±13)
54 (1400)	±5⁄16 (±8)	±9⁄16 (±14)
60 (1500)	±5⁄16 (±8)	±5⁄6 (±16)

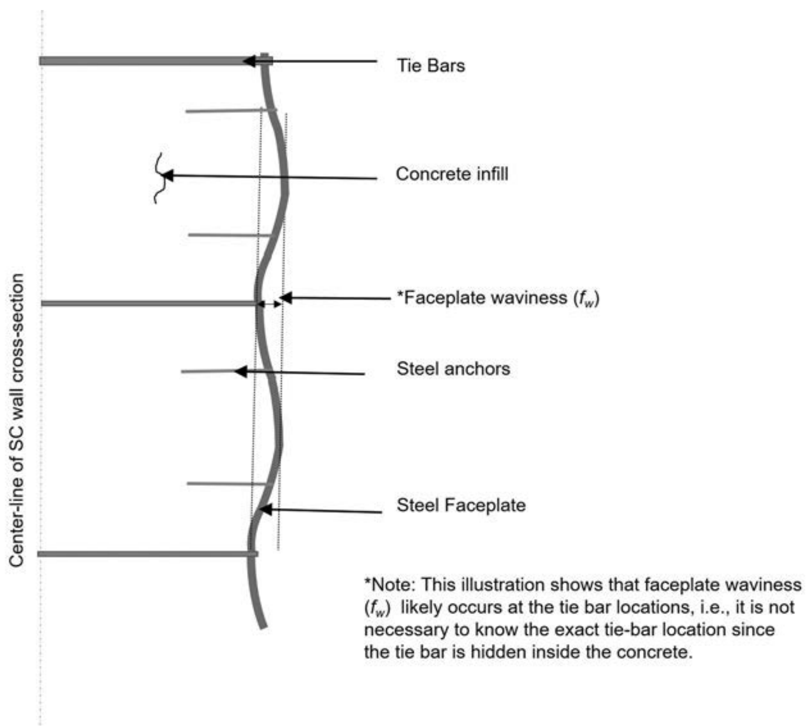


Fig. C-NM2.1. Faceplate waviness. (The faceplate waviness and the variation in tie-bar dimensions has been exaggerated for illustration purposes.)

waviness (imperfections) up to $0.65t_p$ were analyzed. The faceplates developed more than 95% of their yield strength (i.e., $0.95A_sF_y$) at the axial compressive strength. Figure C-NM2.2 was developed using the results of the finite element analyses. It illustrates the compression force (F_steel) carried by the faceplates normalized with respect to its yield strength (A_sF_y) versus the average strain over the length. For nonslender faceplates (e.g., with $s/t_p = 24$, where s is the spacing of steel anchors and t_p is the thickness of faceplate), the reduction in the normalized compression strength of the faceplates is less than 5% for an increase in imperfection from $0.1t_p$ to $0.6t_p$. However, for slender faceplates (e.g., with $s/t_p = 36$) that are not permitted by Appendix N9, Section N9.1.3, this reduction in the normalized compression strength is more substantial and the post-peak behavior is degrading.

NM3. SHOP PAINTING

Add the following:

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

NM4. ERECTION

2. Stability and Connections

Replace section with the following:

Consideration needs to be made for the handling, transportation and erection of an SC wall panel, sub-module or module before it is placed in the erected position. The tolerances for the SC wall are inspected in the fabrication shop and in the erected condition. Since the SC wall assembly is not self-supporting, care should be taken during the transportation and erection of these walls. It is recommended that a formal erection plan be prepared and submitted to the engineer of record.

Add the following new section:

7. Tolerances for Cranes

The CMAA Specification tolerances have been adopted where appropriate. The criteria for column base lines, crane runway girders, and rail eccentricity provide tolerances not prescribed by the CMAA Specification (CMAA, 2015). These additional tolerances, which have evolved in the Nuclear Specification, minimize secondary effects onto the building structure and provide the additional quality control required in a nuclear facility.

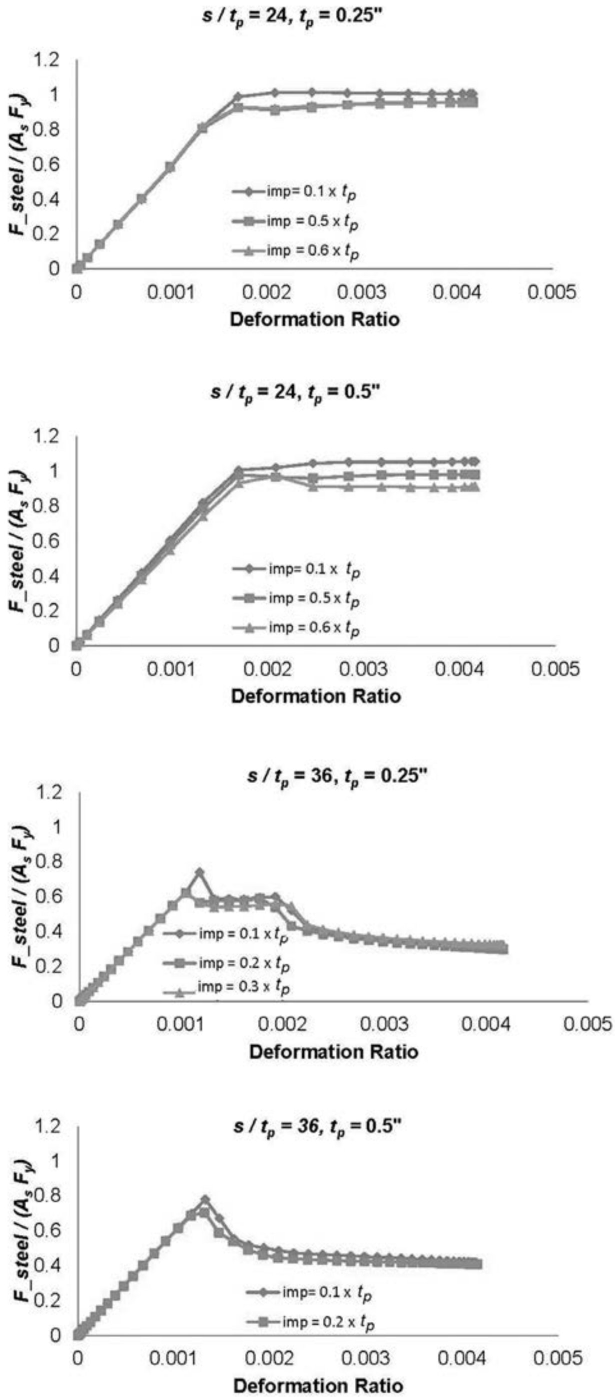


Fig. C-NM2.2. Normalized force carried by faceplates versus average strain.

CHAPTER NN

QUALITY CONTROL AND QUALITY ASSURANCE

Replace Chapter N of the Specification Commentary with the following:

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

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

ASME NQA-1 (ASME, 2015b) stipulates the requirements for the establishment and execution of quality assurance programs for nuclear facilities. Quality assurance programs are pertinent to the designer, engineer, material supplier, fabricator, erector and constructor, and each entity is required to establish such a program. The provisions of the Nuclear Specification are intended to supplement the NQA-1 requirements.

Subpart 2.4 of ASME NQA-1 (ASME, 2015b) establishes installation, inspection and testing requirements for various structural items, including structural steel.

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

There are basic differences between the *Specification* and the Nuclear Specification regarding how quality assurance and quality control are applied to fabricated structural steel. In both the *Specification* and the Nuclear Specification, the quality control functions are performed by the fabricator or erector. In the *Specification*, the quality assurance functions are performed by the authority having jurisdiction (AHJ), applicable building code, purchaser, owner, or engineer of record (EOR). In the Nuclear Specification, the quality assurance functions are performed by the fabricator or the erector as defined in their quality program. The fabricator's or erector's quality program are audited and approved by the owner or their representative.

The owner of the plant will provide surveillance over the fabricator or erector as they perform the quality assurance tasks to ensure they adhere to the design and contractor documents as well as the fabricator's or erector's approved quality program.

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

5b. CJP and PJP Groove Weld NDE

The 10% sampling approach has been added to the 2018 Nuclear Specification. This approach was previously specified in the 1994 edition (AISC, 1994) and is currently being utilized in the construction of new, domestic nuclear plants.

5g. Documentation

Usage of NCIG-01, -02 and -03 is not directly applicable for treatment of impulsive or impactive loads to structural steel members and connections. The engineer of record should justify the usage of these NCIG documents on a case-by-case basis.

NN6. MINIMUM REQUIREMENTS FOR INSPECTION OF COMPOSITE CONSTRUCTION

Add the following:

Properly designed concrete in steel-plate composite (SC) walls is expected to have good consolidation due to the lack of congestion. Specific configurations may increase congestion locally and pose challenges for concrete placement in areas such as connections to walls and slabs, anchorages to basemats, openings, embedment plate anchorages, and other irregularities. The design in areas of congestion should consider constructability and detail the SC walls accordingly. Mock-ups may be employed to demonstrate that a particular construction technique provides adequate quality of concrete placement in SC walls.

Honey combing or void formation can be prevented in SC construction by ensuring proper compaction. As compared to reinforced concrete construction, proper compaction in similar SC construction is easier to achieve due to the absence of reinforcement layers in SC walls.

Table NN6.1 provides inspection requirements for steel elements of composite construction. The various inspection attributes listed in this table were derived from ANSI/SDI QA/QC-2011, *Standard for Quality Control and Quality Assurance for Installation of Steel Deck* (SDI, 2011).

NN7. NONCONFORMING MATERIAL AND WORKMANSHIP

A corrective action report (CAR) is initiated when it is determined by the fabricator or erector that there is a systematic pattern of nonconforming material or workmanship. The CAR will remain open until a root cause has been determined and corrective action taken to make the necessary changes to the process or procedures identified in the root cause analysis. If necessary, this will include changes to the fabricator's or erector's quality assurance program.

APPENDIX N1
DESIGN BY ADVANCED ANALYSIS

Modify Appendix 1 of the Specification Commentary as follows.

N1.3. DESIGN BY INELASTIC ANALYSIS

1. General Requirements

Add the following to the end of the first paragraph:

Relief from thermal load action is best achieved using design features mentioned in the User Note for Sections NB2.5d and NB2.6d. Additionally, the Commentary for these sections mentions analysis approaches, including rigorous second-order analysis accounting for large-displacement theory and catenary behavior, that can provide relief from thermal load effects. As demonstrated by Usmani et al. (2001) and Wang and Yin (2005), formation of a plastic hinge can lead to further relief from thermally induced forces and moments provided that the member's or connection's inelastic deformation capacity is not exhausted. Peer review is recommended in view of the complexities regarding this type of nonlinear inelastic analysis.

APPENDIX N4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

Modify Appendix 4 of the Specification Commentary as follows:

N4.1 GENERAL PROVISIONS

Add the following:

Material properties at elevated temperatures included in the Nuclear Specification cover structural steel commonly used as defined in the *Specification* (AISC, 2016a). For other steels such as stainless steel and forging steel, suitable properties should be obtained based on reliable test results. It should be also pointed out that the material properties at elevated temperatures are short-term properties intended for fire design by analysis only. They should not be used in assessing the long-term performance of structural steel subjected to elevated temperature.

APPENDIX N5

EVALUATION OF EXISTING STRUCTURES

Replace Appendix 5 of the Specification Commentary with the following:

N5.1. GENERAL PROVISIONS

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

N5.2. MATERIAL PROPERTIES

2. Tensile Properties

Using tensile yield strength directly taken from certified material test reports (CMTR) or certified reports for evaluation of existing steel structures is generally not acceptable to the U.S. Nuclear Regulatory Commission (NRC, 2012), because the use of actual yield stress to establish the available strength is not consistent with the nominal yield strength design basis of prior AISC Specifications.

6. Bolts

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

APPENDIX N9

STEEL-PLATE COMPOSITE (SC) WALLS

Nuclear structures involve heavy concrete construction to provide adequate radiation shielding and seismic performance. This results in longer construction durations and large field labor force requirements. Generic modular construction, especially modular steel-plate composite (SC) construction, can minimize schedule and labor requirements. Faceplates on the exterior eliminate formwork and serve as equivalent reinforcement when steel anchors are used.

SC walls are plate or shell-type structures; they are typically not part of frame structures. In SC construction, concrete walls are reinforced with faceplates anchored to concrete using steel anchors and connected to each other using steel ties. The behavior of SC walls under axial tension, axial compression, flexure, and out-of-plane shear is comparable to that of reinforced concrete walls. However, behavior under in-plane shear, combined in-plane forces and out-of-plane moments, and thermal conditions can be significantly different from that of reinforced concrete walls. Additionally, some SC-specific limit states such as faceplate local buckling, interfacial shear failure, section delamination, etc., have to be addressed with adequate detailing of the SC wall section.

This appendix provides specifications for SC walls in safety-related nuclear facilities. The general requirements specify the range of applicability of the specifications and the section detailing requirements to address SC-specific limit states of local buckling, interfacial shear failure, and section delamination. Construction loads have not been addressed in this appendix, as they act on the empty modules. Performance requirements are specified for the connections of SC walls.

This appendix permits the use of stainless steel materials, but the provisions need to be applied judiciously to stainless steel SC walls. The modulus of elasticity and shear modulus of elasticity values for stainless steel are based on the values provided in Table 2-9 of AISC Design Guide 27, *Structural Stainless Steel* (Baddoo, 2013). The values in the Design Guide are taken from ASME *Boiler and Pressure Vessel Code*, Section II: Materials—Part D: Properties (Customary) (ASME, 2015a), with the value for the austenitic stainless steels rounded down to 28,000 ksi (190 000 MPa). Poisson's ratio is taken as 0.3 and the shear modulus of elasticity, G, is taken as $0.385E$.

This appendix applies to design of SC walls and their connections and anchorages. The provisions of the appendix are based on the experimental database discussed in the References. The conservatism of the provisions has also been verified using the experimental database. The appendix is limited to SC walls satisfying the general requirements of Section N9.1.1. The faceplates of the SC walls should be anchored to the concrete infill, and connected to each other using ties. Ties provide structural integrity and prevent delamination of the plain concrete core. The spacing of ties should be less than or equal to the thickness, t_{sc}, of the SC walls.

This appendix is also limited to SC walls with only two faceplates on the exterior surfaces and no additional reinforcing bars. SC walls with more than two steel plates have been used for the design of the primary shield structure [e.g., Booth et al. (2013)], but the specifications in this appendix are not applicable to them. This appendix is not applicable to half SC slabs with only one exterior faceplate. The appendix is not applicable to SC wall piers (with no flange plates). The seismic behavior of SC wall piers is discussed in Epackachi et al. (2014).

Figure C-A-N9.1.1 is provided to facilitate the use of Appendix N9.

N9.1. DESIGN REQUIREMENTS

The design of steel-plate composite (SC) walls needs to be consistent with the intended behavior of the overall structure and the assumptions made in their analysis.

1. General Provisions

(a) The minimum thickness, t_{sc}, for exterior walls is based on Table 1 of the Standard Review Plan (SRP), Section 3.5.3, Revision 3 (NRC, 2007a). It requires minimum 16.9-in.-thick (430 mm) 4-ksi (28 MPa) reinforced concrete (RC) walls to resist a tornado missile. Conservatively, the region is assumed as Region I and the SC wall is treated as a RC wall for missile loading. The minimum thickness for interior walls is based on the maximum reinforcement ratio ($\rho = 0.05$) and minimum faceplate thickness, t_p, equal to 0.25 in. (6 mm).

$$t_{sc}^{min} = 2t_p/\rho = 2\left(0.25 \text{ in.}\right)/0.05 = 10 \text{ in.}$$

$$t_{sc}^{min} = 2t_p/\rho = 2\left(6 \text{ mm}\right)/0.05 = 240 \text{ mm}$$

The specified minimum thickness values are slightly more conservative than the absolute minimums for both exterior and interior SC walls.

The maximum thickness limit is based on the experimental database of out-of-plane shear tests conducted on SC walls in Japan, Korea and the U.S. (Sener and Varma, 2014). SC wall thicknesses greater than 60 in. (1500 mm) are not permitted due to the lack of test data (for in-plane and out-of-plane forces) and possible concerns about the section behaving as a unit (structural integrity). However, recent tests and numerical studies (Booth et al., 2013) on primary shield walls with extremely large thickness [10 to 14 ft (3 to 4.3 m)], consisting of three steel plates (two exterior and one interior), and transverse web plates have confirmed their composite behavior and design strengths.

Experimental and numerical results may be used to justify the applicability and conservatism of this appendix to SC walls thicker than 60 in. (1500 mm).

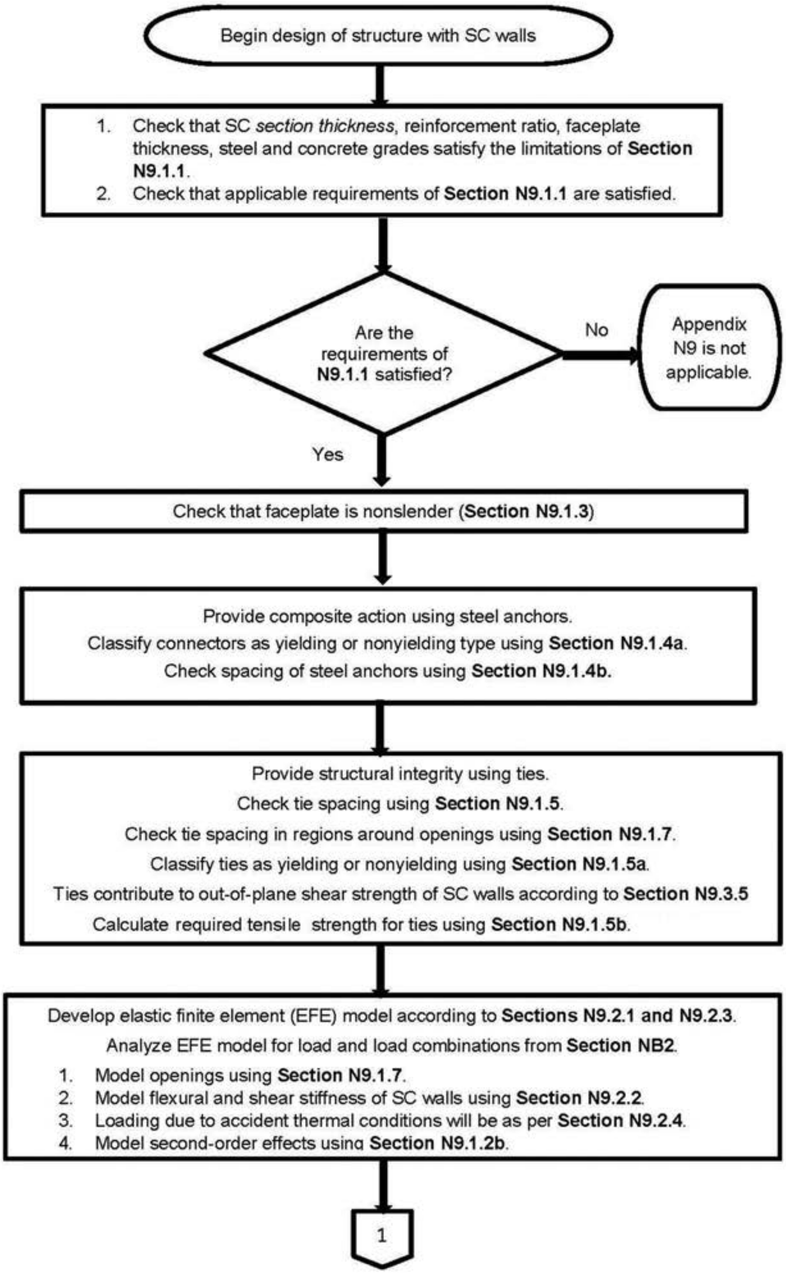
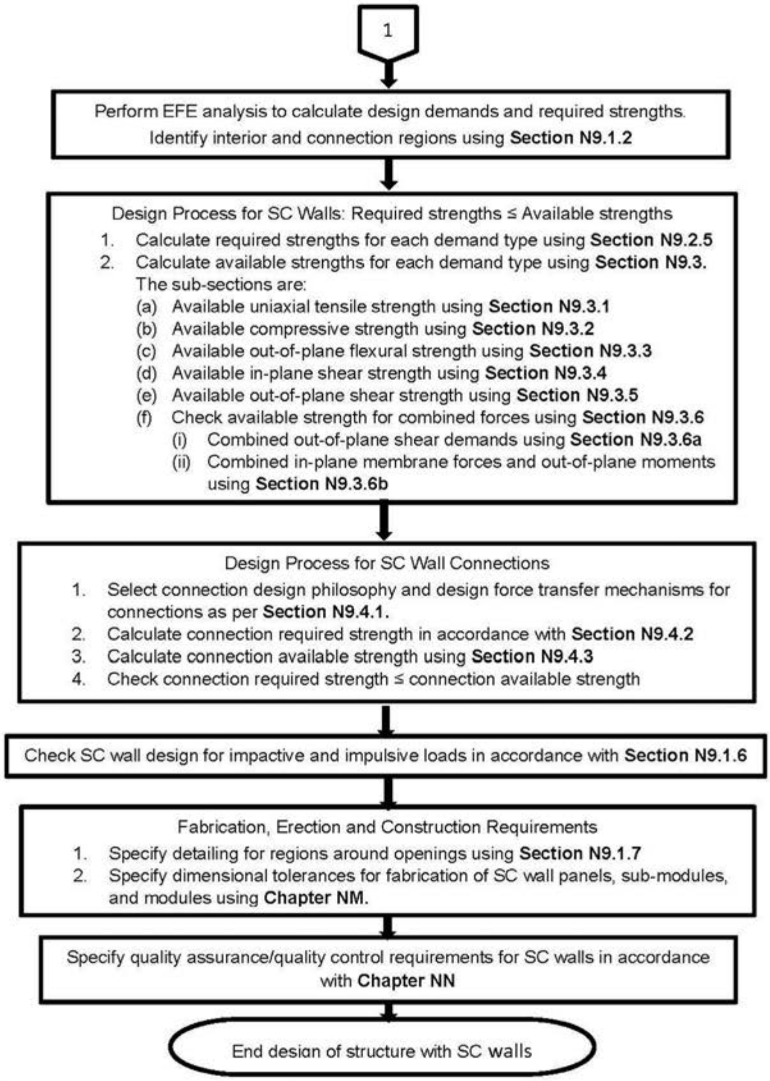


Fig. C-A-N9.1.1. Flowchart to facilitate use of Appendix N9.



General Note:

The elastic finite element model should be made using any system of consistent units. The design demands and required strengths are calculated by performing an elastic finite element analysis. However, before using the equations in this Appendix, the units of the calculated design demands and required strengths should be made consistent with the corresponding units in the Appendix equations. For example, the units for design demands and other material parameters used in the equations of this Appendix are as follows:

(a) The required and available out-of-plane moment strengths are in kip-in./ft (N-mm/m).
(b) The required and available membrane in-plane force strengths, and out-of-plane shear force strengths are in kip/ft (N/m).
(c) The modulus of elasticity for steel and concrete are in ksi (MPa).

Fig. C-A-N9.1.1 (continued). Flowchart to facilitate use of Appendix N9.

(b) Typically, at least 0.25-in.-thick (6 mm) faceplate is needed for adequate stiffness and strength during concrete placement and rigging and handling operations. Additionally, faceplates thinner than 0.25 in. (6 mm) can have the material properties and imperfections (waviness, etc.) of sheet metal (instead of structural plates). The maximum faceplate thickness of 1.5 in. (38 mm) corresponds to the reinforcement ratio of 0.050 for the 60-in.-thick (1500 mm) SC wall. By limiting the faceplate thickness to 1.5 in. (38 mm), preheat will typically not be required.

(c) Use of a very low reinforcement ratio (lower than 0.015) poses concerns regarding handling strength and stiffness in addition to residual stresses due to fabrication operations and concrete casting. The use of very high reinforcement ratios (above 0.050) is not recommended because it can result in higher concrete stresses and change the governing limit state from faceplate yielding to concrete inelasticity and failure in compression, which can reduce the ductility of composite SC walls for in-plane shear loading.

For example, Table C-A-N9.1.1 shows the principal stresses in concrete and steel due to pure in-plane shear loading calculated using the mechanics based model presented by Varma et al. (2014). The table was developed for SC walls with 36 in. (900 mm) concrete thickness, f_c' = 6 ksi (41 MPa), and faceplates with F_y = 50 ksi (350 MPa). As shown, the concrete minimum principal compressive stress (σ_{c-p2}) changes from $-0.15f_c'$ to $-0.35f_c'$ as the reinforcement ratio increases from 0.015 to 0.050. The upper limit of 0.050 for reinforcement ratio is based on this in-plane shear behavior and the lack of additional experimental data for very high reinforcement ratios.

(d) A minimum yield stress of 50 ksi (350 MPa) is specified for the faceplates to prevent: (i) residual (locked-in) stresses from concrete casting, and (ii) thermally induced stresses from causing premature yielding and limiting the strength or ductility of the SC walls. For example, if the temperature increase of 230°F (128°C) is fully restrained, the corresponding strain will exceed the yield strain of ASTM A36 steel. Additionally, high-strength steels with yield stress greater than 65 ksi (450 MPa) are typically less ductile, and hence not desirable for beyond-safe shutdown earthquake shaking.

(e) The requirements for proportioning and selecting the constituents used in concrete mix design are defined in ACI 349-13 or ACI 349M-13 (ACI, 2013). The use of concrete with strength less than 4 ksi (28 MPa) is rare in safety-related nuclear facilities with the possible exception of base mats. The minimum concrete strength of 4 ksi (28 MPa) is also specified so that the minimum principal (compressive) stress in concrete remains in the elastic range while faceplate yielding occurs under in-plane shear loading.

The provisions of this appendix are based on the test results of specimens with a specified compressive strength of concrete of 8 ksi (55 MPa) or less. Figure C-A-N9.1.2 presents the range of concrete compressive strength from the experimental database for out-of-plane shear tests. The figure is based on the dataset

TABLE C-A-N9.1.1
Principal Stresses Due to In-Plane Shear Loading at Yield Load

Reinforcement Ratio	Face-plate Thickness	Section Thickness	Yield Load	Steel Principal Stress		Concrete Principal Stress	
ρ	t_p	t_{sc}	V_{ni}	$\sigma_{s\text{-}p1}$ (max.)	$\sigma_{s\text{-}p2}$ (min.)	$\sigma_{c\text{-}p1}$ (max.)	$\sigma_{c\text{-}p2}$ (min.)
	in. (mm)	in. (mm)	kip/in. (N/mm)	ksi (MPa)	ksi (MPa)	ksi (MPa)	ksi (MPa)
0.015	0.27 (7)	36.5 (930)	28.8 (5000)	53.3 (370)	7.44 (51)	0 (0)	–0.910 (–6.3)
0.020	0.36 (9)	36.7 (930)	37.7 (6600)	52.3 (360)	4.95 (34)	0 (0)	–1.14 (–7.9)
0.030	0.54 (14)	37.1 (940)	54.4 (9500)	50.4 (350)	0.730 (5)	0 (0)	–1.53 (–11)
0.040	0.72 (18)	37.4 (950)	70.0 (12000)	48.6 (340)	–2.66 (–18)	0 (0)	–1.84 (–13)
0.050	0.90 (23)	37.8 (960)	84.7 (15000)	47.1 (320)	–5.42 (–37)	0 (0)	–2.08 (–14)

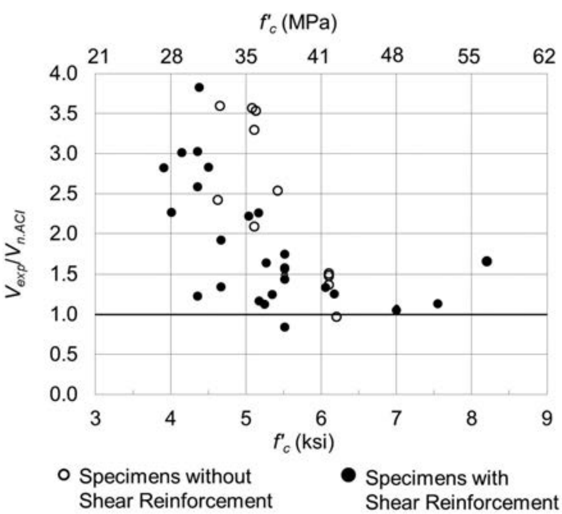


Fig. C-A-N9.1.2. Range of concrete compressive strength from experimental database.

discussed in Sener and Varma (2014). The applicability of the various requirements and provisions of this appendix needs to be verified for SC wall designs with specified compressive strength of concrete greater than 8 ksi (55 MPa).

The use of lightweight concrete is not permitted due to lack of experimental data for SC walls constructed using lightweight concrete.

(f) The detailing requirement of Section N9.1.3 prevents the SC specific limit state of faceplate local buckling from occurring before yielding in compression.

(g) The detailing requirements of Section N9.1.4 provide adequate steel anchors to anchor the faceplates to the concrete infill. The steel anchors are designed to (i) develop the yield strength of the faceplate over a distance of no more than three times the section thickness, and (ii) prevent interfacial shear failure from occurring before out-of-plane shear failure.

(h) The detailing requirements of Section N9.1.5 provide adequate ties to prevent section delamination through the plain concrete infill. The ties also serve as out-of-plane shear reinforcement and ensure structural integrity during rigging and concrete placement.

(i) The requirement for the effective rupture strength per unit width to be greater than the yield strength per unit width ensures that gross yielding of the faceplates with holes governs over net section rupture.

(j) The majority of the experimental investigations have been performed on SC walls with faceplates that have the same nominal thickness and specified minimum yield strength. The lack of uniformity between the yield strength of the two faceplates exacerbates the potential for section delamination through the plain concrete. The requirements of Appendix N9, Section N9.1.5, consider delamination due to 50% nonuniformity between the faceplate yield strengths (thickness × yield stress). However, Appendix N9, Section N9.1.1, conservatively stipulates that the specified minimum yield strength and faceplate thickness be identical for both faceplates.

(k) Steel ribs may be welded to the faceplates of SC walls to increase the stiffness and strength of the empty modules. This increased stiffness improves the behavior of the empty modules during transportation, handling and erection. The ribs also improve the resistance of the faceplates to hydrostatic pressure from concrete casting. After concrete hardening, the ribs prevent local buckling of the faceplates. Therefore, when used in SC walls, these steel ribs should be welded to the faceplates to fully develop the yield strength of their connected element (leg). As shown in Figure C-A-N9.1.3, the embedment of the steel ribs into the concrete is limited to (i) prevent the use of large depth steel ribs that can alter the mechanics of the SC wall behavior, and (ii) minimize the interference of ribs on the performance of the other steel anchors. However, the contribution of steel ribs is not considered for any design parameters (e.g., composite action, available strengths, etc.).

(l) Faceplate splices are detailed to ensure that the limit state of gross section yielding governs.

Vent Holes. The faceplates of SC walls are connected to each other using ties. According to Section N9.1.5, these ties have spacing less than or equal to the section thickness, t_{sc}. The tensile force requirements for these ties are provided in Section N9.1.5b to prevent section delamination failure. Additionally, the faceplates are anchored to the concrete infill in between tie locations using steel anchors. The spacing requirements for steel anchors are provided in Section N9.1.4b. The internal steam pressure associated with evaporation of water from the concrete infill due to elevated temperatures from accident conditions can be resisted by the steel structure consisting of faceplates, ties and steel anchors, without significant stress. Additional vent holes or weep holes for release of steam pressure due to accident thermal conditions are not required. Additionally, the use of vent holes or weep holes is impractical for SC walls used in liquid or water storage tanks, where the faceplates may be in direct contact with hot water during accident conditions.

Curved SC Walls. The appendix was developed for straight SC walls. If the SC walls in application have any curvature, effects of curvature on detailing and design of SC walls need to be evaluated. This is necessary as there is no specific data available for curved SC walls at present. For the ratio of radius of curvature-to-section thickness values greater than 20, the effects of curvature may turn out to be negligible, and the provisions of the appendix will be adequate. However, for the ratio of radius of curvature-to-section thickness values less than 20, project specific design and detailing requirements for SC walls seem to be warranted.

Alternate design methods for SC walls not meeting the general provisions may be based on (i) project-specific large-scale test data, or (ii) results of nonlinear inelastic analyses conducted using modeling approaches that are benchmarked against applicable test data and peer-reviewed. Alternatively, subject to peer-review, the wall design may also be performed in accordance with ACI 349-13 or ACI 349M-13 provided that (i) the faceplate thickness and its composite action is minimized to primarily enable it to function as formwork, (ii) conventional rebar is provided to develop adequate section strength for demands due to in-plane and out-of-plane forces and moments, and (iii) the faceplates are evaluated for stresses and strains due to strain compatibility to ensure that they remain below their yield and local buckling threshold [similar to the design of liner plates in concrete containment structures according to ACI 359-01 (ACI, 2001)].

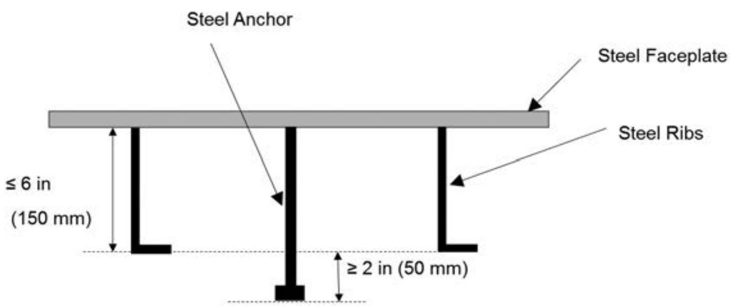


Fig. C-A-N9.1.3. Embedment depth of steel ribs.

2. Design Basis

Safety-related nuclear facilities, for example containment internal structures, consist of labyrinthine walls that are connected to each other and anchored to the concrete basemat. Force transfer between walls occurs at connections and the anchorage to the basemat. To facilitate design, the expanse of SC walls is notionally divided into interior regions and connection regions. Force transfer between SC walls, and composite action between faceplates and concrete, develops over connection regions. Figure C-A-N9.1.4 illustrates the typical interior and connection regions for SC walls.

The requirement for connection regions to be less than or equal to wall thickness ($\leq 2t_{sc}$) is based on typical development lengths of No. 11 to No. 18 reinforcing bars, which are used typically in nuclear construction. Specifying connection region lengths less than the wall thickness ($\leq t_{sc}$) can be impractical and lead to detrimental congestion of steel anchors and tie bars. Connection regions are designed to achieve adequate force transfer and composite action in accordance with the requirements of Section N9.4.

2a. Required Strength

Seismic analyses of safety-related nuclear facilities are typically conducted in two steps: (1) dynamic soil structure interaction analyses; and (2) subsequent equivalent static or dynamic analyses of the structure only (Varma et al., 2014). The load combinations imply linear superposition of the required strengths. Other methods of analysis have been ruled out because the finite element method is the only practically feasible method for global analysis of continuum structures. As discussed in Section N9.1.6c, additional dynamic analyses may be needed to determine the response of structures to impactive or impulsive loads. This is characteristic of structural design of safety-related nuclear facilities and comparable to ACI 349-13 or ACI 349M-13, Appendix F (ACI, 2013), and also to Section NB3.14 of the Nuclear Specification.

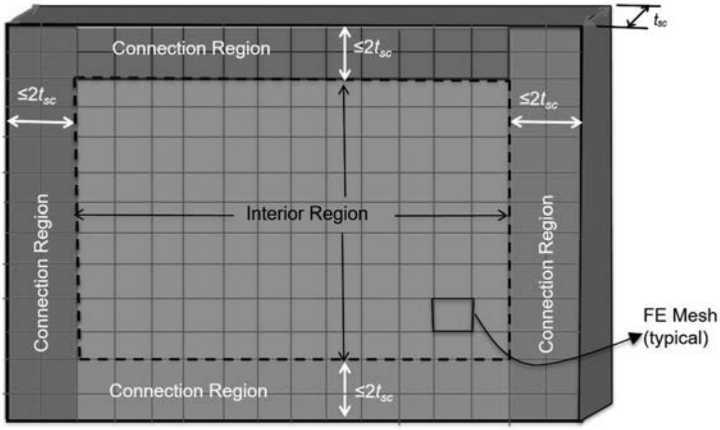


Fig. C-A-N9.1.4. The expanse of SC walls separated into connection regions and interior regions.

Because the analysis is elastic, the thermal demands will be combined with demands due to mechanical loads using appropriate load combinations. The load combinations for operating thermal and seismic do not consider concrete cracking. However, concrete cracking is considered in accident thermal and seismic. Because concrete is considered cracked for both mechanical and thermal loads, the demands due to these loads are linearly superimposed.

2b. Design for Stability

The thickness of SC walls in nuclear applications will generally exceed 2 ft (0.6 m). Their typical height-to-thickness ratios will meet the requirements of ACI 318-14 or ACI 318M-14, Section 6.2.5(b) (ACI, 2014). Second-order analysis will generally be unnecessary for the labyrinthine structures where SC walls will be used. In the rare situation that the ACI requirements are not satisfied, the structure will generally meet the limitations of *Specification* Appenix 7, Section 7.3, allowing first-order analysis to be performed with notional lateral loads in lieu of second-order analysis. Second-order analysis by the direct analysis method is limited to steel frame structures with linear (beam, column) elements. It is not applicable to labyrinthine structures made up of SC or RC walls.

3. Faceplate Slenderness Requirement

Local buckling of faceplates is an SC specific limit state. The faceplates are required to be nonslender, i.e., yielding in compression must occur before local buckling. When subjected to compressive stresses, the faceplate undergoes local buckling between the steel anchors as shown in Figure C-A-N9.1.5. As shown, the horizontal lines joining the steel anchors (or ties) act as fold lines and local buckling occurs between them. The buckling mode indicates fixed-ends along the vertical lines with steel anchors and partial fixity along the vertical lines between steel anchors.

Experimental studies have been conducted to evaluate the effects of plate slenderness ratio, s/t_p, defined as the steel anchor spacing, s, divided by the plate thickness, t_p,

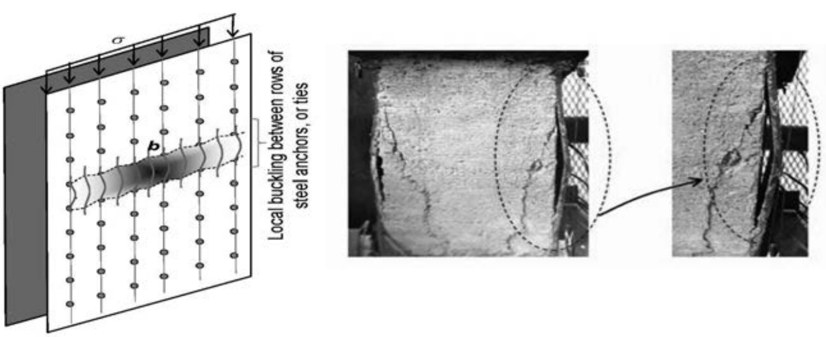


Fig. C-A-N9.1.5. The buckling mode of the faceplate (Zhang et al., 2014).

on local buckling of faceplates. Zhang et al. (2014) have summarized these experimental studies and conducted additional numerical analyses to confirm and expand the experimental database. Figure C-A-N9.1.6 from Zhang et al. (2014) shows the relationship between the normalized critical buckling strain (buckling strain/steel yield strain, $\varepsilon_{cr}/\varepsilon_y$) and the normalized faceplate slenderness ratio ($s/t_p \times F_y/E$). As shown, ε_{cr} is reasonably consistent with Euler's curve with a partially fixed ($K = 0.7$) end condition. Also, no data point falls in the shaded area, implying yielding occurs before local buckling for a normalized plate slenderness ratio less than 1.0. Because ties may also act as steel anchors, Equation A-N9-2 considers the largest unsupported length between rows of steel anchors or ties, b.

The faceplate slenderness equation (Equation A-N9-2) will be slightly more conservative for stainless steel plates because of the lower elastic modulus value for stainless steel. For faceplates with a specified minimum yield stress greater than or equal to 50 ksi (350 MPa), no additional limits are placed on locked-in stresses or displacements due to concrete casting. The use of faceplates with a specified minimum yield stress less than 50 ksi (350 MPa) is not permitted because:

(a) The potential for local buckling before yielding becomes higher for lower yield stress faceplates due to the higher proportion of locked in stresses and displacements from concrete casting.

(b) The potential for local yielding due to accident thermal loading conditions becomes higher for lower yield stress faceplates.

4. Requirements for Composite Action

4a. Classification of Steel Anchors

The steel anchors used in SC construction may consist of steel headed stud anchors, embedded steel shapes, or tie bars (smooth or deformed), etc., that can be attached to the faceplates with structural welding or bolting. The shear strength of connectors governs the composite action, interfacial shear strength, and slip between faceplates and concrete infill (Zhang et al., 2014).

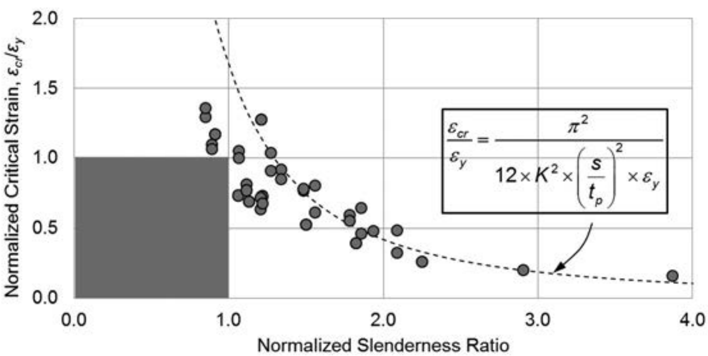


Fig. C-A-N9.1.6. The relationship between buckling strain of plate and normalized slenderness ratio (Zhang et al., 2014).

Steel anchors that have a ductile shear force-slip behavior can redistribute the interfacial shear equally over several connectors. Such connectors are referred to as yielding-type, e.g., steel headed stud anchors. Steel anchors that have a nonductile shear force-slip behavior cannot redistribute the interfacial shear equally over several connectors. Such anchors are referred to as the nonyielding type.

An interfacial slip capability of at least 0.20 in. (5 mm) before reduction in shear strength to 90% of the available shear strength is required to qualify as a yielding-type connector (Figure C-A-N9.1.7). Steel anchors not meeting this requirement are classified as a nonyielding type. Steel headed stud anchors are typically capable of sustaining at least 0.20 in. (5 mm) of interfacial slip in a ductile manner (Ollgaard et al., 1971). All other types of steel anchors need to be tested to determine their available shear strength and interfacial slip capability. An adequate number of tests need to be performed to ascertain the available strength of nonyielding steel anchors. The safety factors applicable for nonyielding steel anchors can be obtained from the experimental studies by following the reliability analysis procedures used by Pallares and Hajjar (2010) and defined by Ravindra and Galambos (1978).

Where a combination of yielding steel anchors and nonyielding steel anchors is used, the maximum strengths of the connectors can't be directly combined. In this case, the system is treated as nonyielding. Therefore the strength of yielding steel anchors is limited to strength corresponding to the interfacial slip at which the nonyielding steel anchors reach their ultimate strength. This is illustrated in Figure C-A-N9.1.8. The strength of the steel anchor system will be the sum of the strengths of individual steel anchors.

Development length, L_d, is the length over which the faceplate can develop its yield strength in axial tension (Zhang et al., 2014). It is similar to rebar development length in RC structures. The development length, L_d, should be designed to be approximately two to three times the wall thickness, t_{sc}, which is the typical development length for No. 11 to No. 18 rebars in reinforced concrete structures.

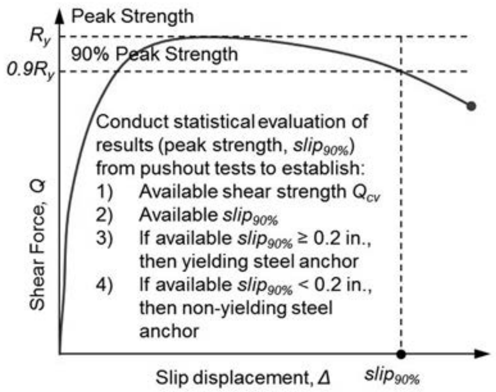


Fig. C-A-N9.1.7. Typical steel anchor force-slip behavior from pushout tests.

4b. Spacing of Steel Anchors

Figure C-A-N9.1.9 from Zhang et al. (2014) shows the free body diagram that resulted in the spacing requirement for yielding steel anchors to achieve faceplate yielding over the development length, L_d. As shown in Figure C-A-N9.1.10, all the yielding steel anchors in the development length contribute equally to developing the yield strength of the faceplate.

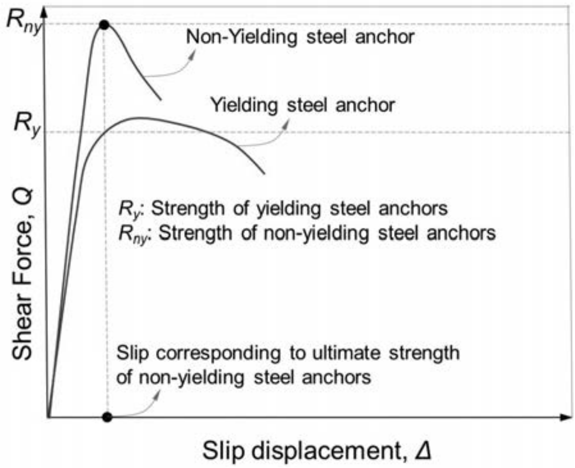
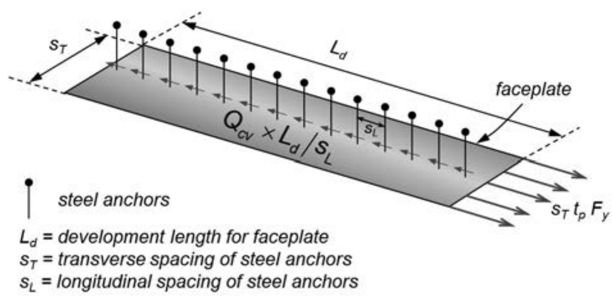


Fig. C-A-N9.1.8. Strength of yielding steel anchors that form a part of nonyielding steel anchor systems.



$$\phi Q_n \frac{L_d}{s_L} \geq s_T t_p F_y$$

$$\text{Therefore, } \frac{\phi Q_n L_d}{t_p F_y} \geq s_T s_L \text{ and } s \leq \sqrt{\frac{\phi Q_n L_d}{t_p F_y}}.$$

Fig. C-A-N9.1.9. Yielding steel anchor spacing requirement for plate yielding.

The interfacial shear strength of SC walls is specified to be greater than the corresponding out-of-plane shear strength of SC walls. This prevents interfacial shear failure, which is an SC specific limit state similar to bond shear failure in reinforced concrete, from governing the behavior and failure mode. Figure C-A-N9.1.11 shows the free body diagram that resulted in the spacing requirement for yielding steel anchors so that out-of-plane shear failure would occur before interfacial shear failure.

Figure C-A-N9.1.11(a) shows the derivation of the spacing requirement for steel anchors for preventing interfacial shear failure from occurring before out-of-plane shear failure. The figure shows the free body diagram for a length, L_v, of the composite wall subjected to out-of-plane shear loading. As shown, the out-of-plane shear, V, produces a change in the bending moment, ΔM, along the shear span, L_v. The tension forces on the bottom faceplate are calculated by dividing the moment, M or $M + \Delta M$, by the effective arm length, jt_{sc}. The spacing of the shear connectors in the longitudinal direction is s_L, and the spacing in the transverse direction is s_T.

Figure C-A-N9.1.11(b) shows the free body diagram of the bottom faceplate in tension over the length, L_v. The tension forces resulting from the applied moments are included in the figure. The equilibrating force from the yielding steel anchors is calculated as the design shear strength, Q_{cv}, of each connector multiplied by the number of connectors. The largest possible shear force, V, is equal to the nominal out-of-plane shear strength, V_{no}. Therefore, interfacial shear failure will not occur before out-of-plane shear failure as long as the steel anchor spacing, s, satisfies the Equation in Figure C-A-N9.1.11(c).

For nonyielding steel anchors, the resistance is not divided equally between all connectors. Instead, a triangular distribution occurs with the maximum value for the first or last connector as illustrated in Figure C-A-N9.1.10. This change in the resistance of nonyielding steel anchors results in the changes in the spacing requirements for nonyielding steel anchors.

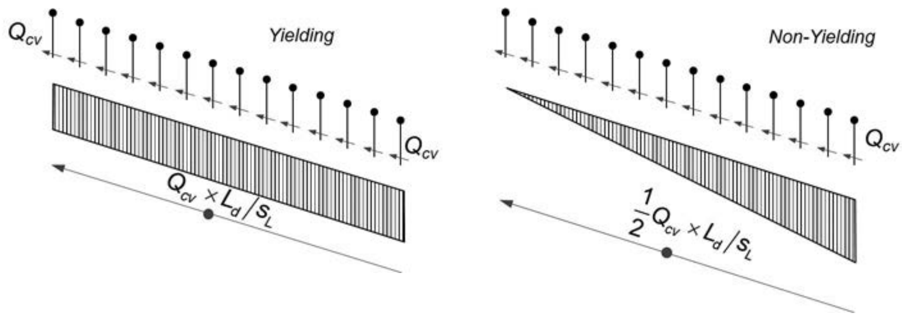
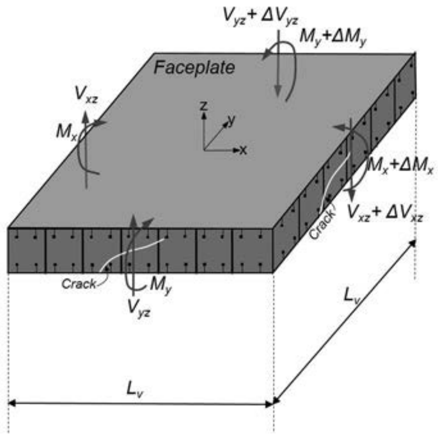
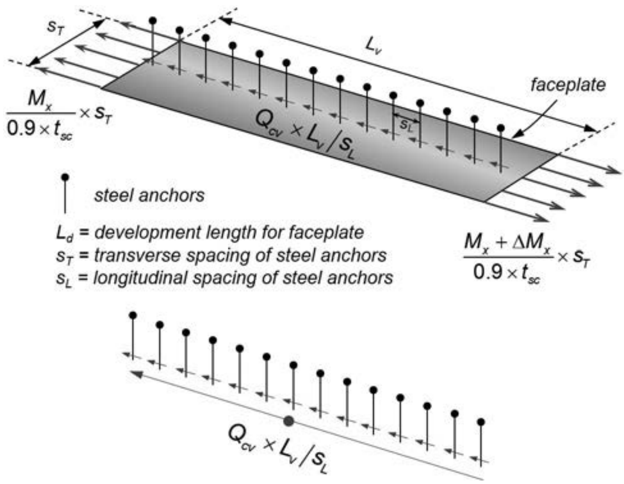


Fig. C-A-N9.1.10. Interfacial shear resistance of yielding and nonyielding steel anchors (LRFD).



(a) Spacing requirement for steel anchors



(b) Free body diagram of the bottom faceplate in tension

$$Q_{cv} \frac{L_v}{s_L} \geq \frac{\Delta M_x}{j t_{sc}} s_T$$

Therefore, $s_T s_L \leq Q_{cv} \left(\frac{L_v}{\Delta M_x} \right) 0.9 t_{sc}$

and $\frac{\Delta M_x}{L_v} = V \leq V_{no}$

Substituting these expressions:

$$s \leq \sqrt{\frac{Q_{cv}(0.9 t_{sc})}{V_{no}}}$$

(c) Derivation of equation to avoid interfacial shear failure prior to out-of-plane shear failure

Fig. C-A-N9.1.11. Steel anchor spacing requirement for preventing interfacial shear failure before out-of-plane shear failure (LRFD).

5. Tie Requirements

The ability of the faceplates of SC walls to interact with each other through the concrete infill is very important. This connectivity is required for the SC section to act as an integral composite unit with the two faceplates and the concrete acting in unison. There is a potential failure plane through the plain concrete thickness that can result in delamination or splitting failure of the wall section.

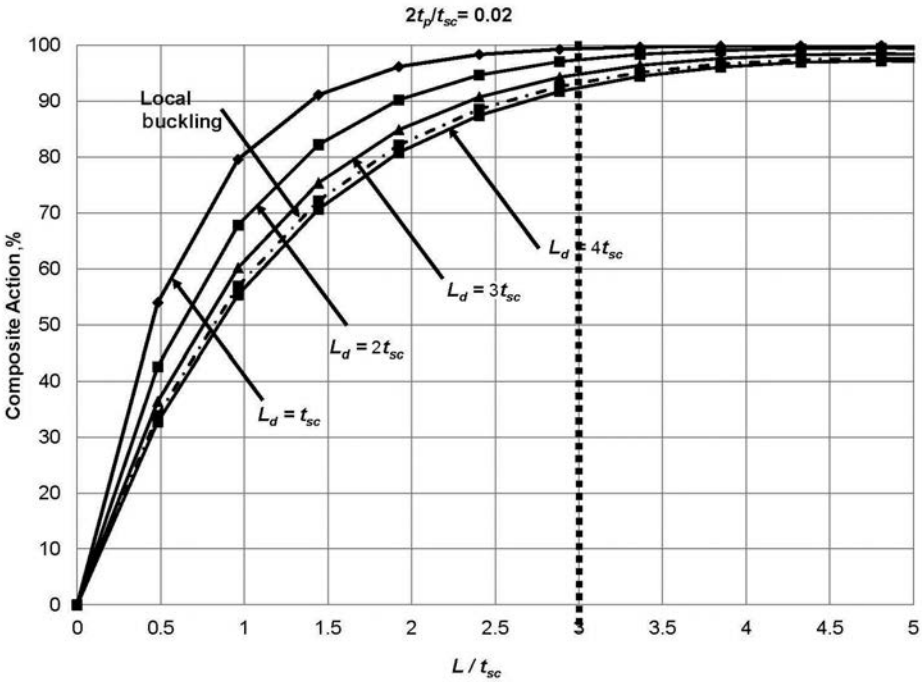
Ties contribute to the out-of-plane shear strength and structural integrity of SC walls. Their contribution to the out-of-plane shear strength (according to Section N9.3.5) may be required for the calculated design demands (required strengths). Ties also provide structural integrity in terms of resistance to delamination or splitting failure due to eccentricities within the section in the force transfer region or due to disparity between the faceplate strengths. Ties may participate in force transfer mechanisms in connection regions of SC walls. Tie spacing can be as large as section thickness, t_{sc}, or 48 times the tie bar diameter (in accordance with ACI 318 or ACI 318M, Section 25.7.2.1). Ties can be made of any shape and a variety of steel materials permitted in Chapter NA.

The transfer length, L_{TR}, is defined as the length required to develop 100% strain compatibility between the steel and concrete portions of the composite section if only one of the portions (e.g., concrete) is loaded at the end. Zhang et al. (2014) have analytically investigated the potential transfer lengths for composite SC walls subjected to axial loading on the concrete only at the ends. As shown in Figures C-A-N9.1.12(a) and C-A-N9.1.12(b), strain compatibility (steel strain/concrete strain) or the percentage of composite action increases with distance from the concrete only loaded ends. The transfer lengths are typically greater or equal to at least three times the section thickness, t_{sc}, for SC walls with reinforcement ratios of 0.015 to 0.050.

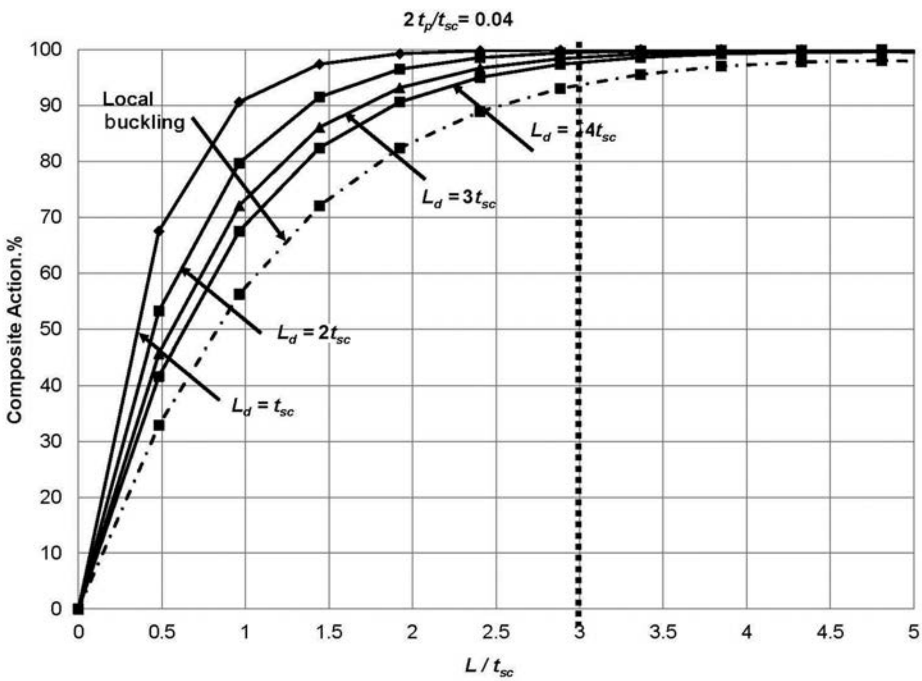
Zhang et al. (2014) show that SC walls designed with steel anchor spacing, s, to satisfy the nonslenderness requirement, and to achieve development lengths, L_d, less than or equal to three times the wall thickness, have transfer lengths, L_{TR}, greater or equal to three times the wall thickness. It is important to note that the development length, L_d, is associated with the shear strength of steel anchors, and their ability to develop the yield strength of the faceplate. The transfer length, L_{TR}, is associated with the relative stiffness (force-slip behavior) of the steel anchors, and their ability to develop strain compatibility between the faceplates and concrete infill. The transfer lengths are longer than the development lengths for typical SC wall designs (faceplates and steel anchor size and spacing).

However, the effects of having longer transfer lengths are somewhat inconsequential. The design capacities or available strengths of SC walls depend on developing the yield strength of the faceplates, not strain compatibility. The effective stiffness of the composite section depends on strain compatibility; however, the effects of having longer transfer lengths and 75 to 90% composite action on the effective stiffness are marginal (Zhang et al., 2014).

The transfer length, L_{TR}, used in the ties strength and spacing requirements is limited to three times the section thickness. Smaller values are improbable and larger values reduce the required force, F_{req}, that the ties have to be designed for.



(a)



(b)

Fig. C-A-N9.1.12. Development of strain compatibility with distance from member end (Zhang et al., 2014).

5a. Classification of Ties

The available tensile strength of ties considers the limit states of (1) gross yielding of ties, (2) net section fracture of ties, and (3) fracture failure of tie-to-faceplate connections. If the limit state of gross yielding governs, the ties are considered as yielding, otherwise the ties are considered as nonyielding. However, there may be cases where components that appear to be governed by yielding may in fact be controlled by nonyielding limit states. Therefore, a minimum margin has been specified between yielding and nonyielding limit state strengths. The requirements of this section ensure that for ties to be classified as yielding shear reinforcement, their nominal rupture strength (or the nominal strength of associated connections) should be at least 1.25 times the nominal yield strength. This information is used in Section N9.3.5 to compute the out-of-plane shear strength.

5b. Required Tensile Strength for Ties

There are two cases where an eccentric moment on the SC wall may cause a splitting failure. Case 1 is when the load is applied to concrete only, and the moment is resisted by the composite section.

If the compressive forces are applied only to the concrete, they will slowly transfer over to the composite section over the transfer length, L_{TR}. Figures C-A-N9.1.13 and C-A-N9.1.14 illustrate the forces in the composite section. However, over this transfer length, there will be an eccentric moment, M_o, that will have to be resisted by the cross section without splitting. The resisting moment, M_R, is depicted in Figure C-A-N9.1.15.

Figure C-A-N9.1.13 considers a lateral section of the wall length along the transfer length, L_{TR}. Figure C-A-N9.1.14 establishes that there is an eccentric moment, M_o, resulting from the significant thickness, t_{sc}, of the wall, and the fact that the force applied on the lefthand side and the resultant on the righthand side are not collinear. The figure includes a calculation of M_o, produced at the mid-thickness of the SC wall.

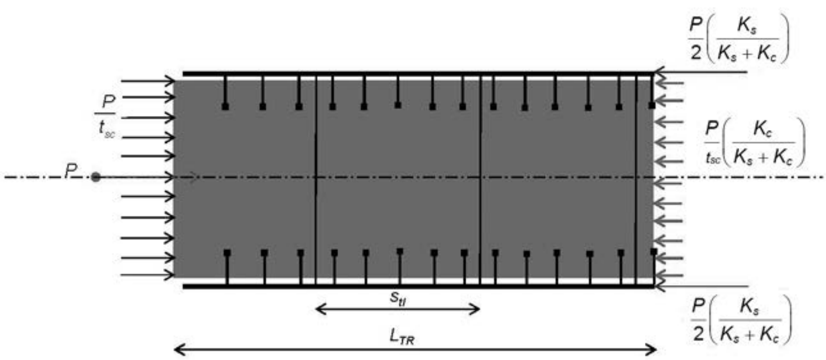
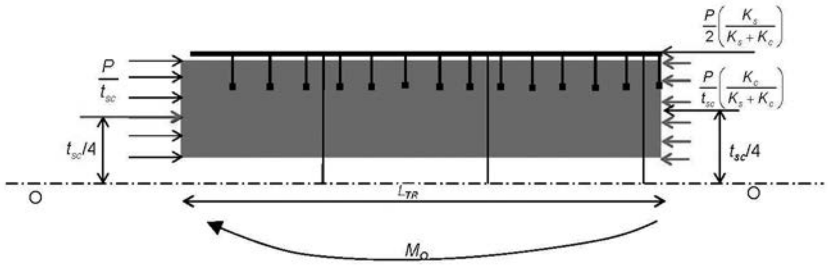


Fig C-A-N9.1.13. Load applied to concrete only, resisted by composite section.



$$+\sum M_{O\text{-}O} = -\left(\frac{P}{t_{sc}}\times\frac{t_{sc}}{2}\times\frac{t_{sc}}{4}\right)+\frac{P}{2}\left(\frac{K_s}{K_s+K_c}\right)\times\frac{t_{sc}}{2}+\frac{P}{t_{sc}}\left(\frac{K_c}{K_s+K_c}\right)\times\frac{t_{sc}}{2}\times\frac{t_{sc}}{4}-M_O=0$$

$$\therefore M_O = -\left(\frac{Pt_{sc}}{8}\right)+\frac{Pt_{sc}}{4}\left(\frac{K_s}{K_s+K_c}\right)+\frac{Pt_{sc}}{8}\left(\frac{K_c}{K_s+K_c}\right)$$

$$\therefore M_O = \frac{Pt_{sc}}{8}\times\left(\frac{K_s}{K_s+K_c}\right)$$

$$\therefore M_O = \frac{P}{2}\times\left(\frac{K_s}{K_s+K_c}\right)\times\frac{t_{sc}}{4} = \text{steel plate force}\times\frac{t_{sc}}{4}$$

Fig C-A-N9.1.14. Eccentric moment, M_O*, acting on the split section.*

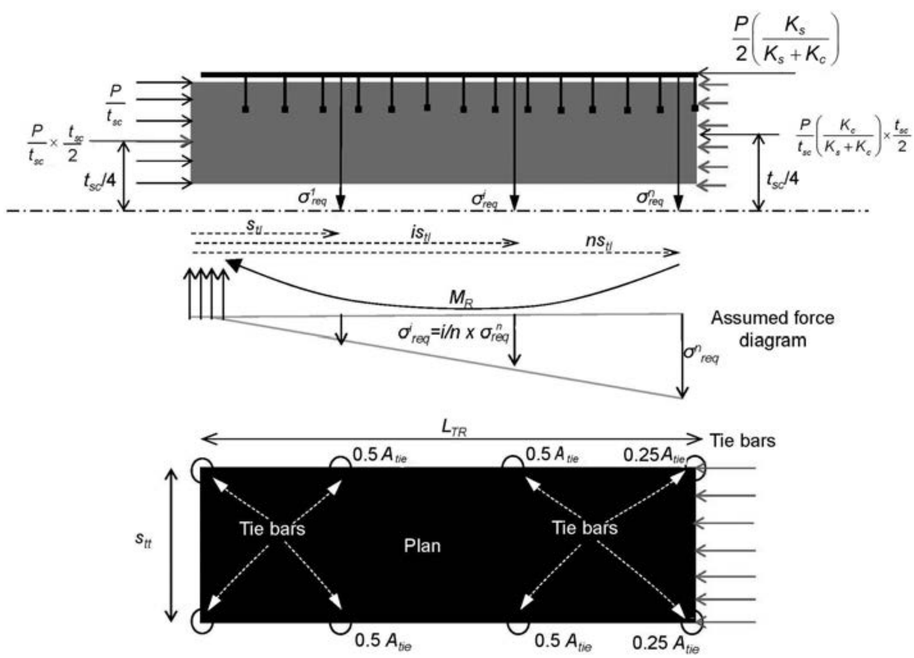


Fig C-A-N9.1.15. Resisting moment, M_R.

Figure C-A-N9.1.15 shows how the eccentric moment, M_o, is resisted by the tie bars with area, A_{tie}, acting along with the concrete in compression. As shown, the strain diagram is assumed to be linear, but the contribution of the concrete to resist tensile stresses is conservatively neglected. The size of the concrete compression block is also assumed to be very small in order to simplify calculations, and the contribution of the concrete compression block to the resisting moment, M_R, is also conservatively ignored. As shown by the plan view in Figure C-A-N9.1.15, a unit portion of the wall with contributing ties is considered. The resisting moment, M_R, is calculated by including the contributions of all the ties in the unit portion.

The required tie strength, F_{req}, is estimated by setting the resisting moment, M_R, greater than or equal to the eccentric moment, M_o. The largest value for the eccentric moment, M_o, is equal to the faceplate force, $(A_sF_y)(t_{sc}/4)$. Based on the study by Zhang et al. (2014) discussed in Commentary Appendix N9, Section N9.1.5, a transfer length value of $3t_{sc}$ has been used in the formulation of Equation A-N9-6.

For example:

t_p = ½ in. (13 mm)
F_y = 50 ksi (350 MPa)
t_{sc} = 30 in. (750 mm)
$s_{tt} = s_{tl} = t_{sc}$

Therefore, from Equation A-N9-6:

$$F_{req} = \left[\frac{(\text{½ in.})(50\text{ ksi})(30\text{ in.})}{4}\right](1)\left[\frac{6}{18(1)^2+1}\right] = 59.2\text{ kips}$$

$$F_{req} = \left[\frac{(13\text{ mm})(350\text{ MPa})(750\text{ mm})}{4}\right](1)\left[\frac{6}{18(1)^2+1}\right] = 269\,000\text{ N} \quad \text{(S.I.)}$$

It is important to note that the required force, F_{req}, is a hypothetical demand that has been posited to evaluate the structural integrity and splitting failure of the section. It is not a real force demand that needs to be deducted from the available capacity of the tie. Additionally, another case is when there is an imbalance in the forces in the thick composite cross section due to different areas and yield strengths of the faceplates. The ties have to provide structural integrity and prevent splitting failure. For example, under in-plane shear loading, the composite section typically develops the yield strength of the section, which could imply slightly different yield forces in the faceplates due to differences in their actual areas or yield stresses (the appendix requires the faceplates to have the same nominal thickness and yield strength).

6. Design for Impactive and Impulsive Loads

This sub-section is based on ACI 349-13 or ACI 349M-13, Appendix F, Special Provisions for Impulsive and Impactive Effects. The definitions of impactive and impulsive loads have also been taken from ACI 349-13 or ACI 349M-13 (ACI, 2013). However, the deformation limits and design criteria given in this section are for SC walls. Impactive and impulsive loads must be considered concurrent with other loads (e.g., dead and live load) in determining the required resistance of structural elements. Evaluation of aircraft missiles is outside the scope of this appendix. It is addressed in NEI 07-13 (NEI, 2011).

6b. Ductility Ratios

Plastic hinge rotation capacity need not be checked if the deformation limit is kept to under 10 for flexure-controlled sections (Varma et al., 2011c). Using Equation A-N9-8 or A-N9-8M to calculate the effective flexural stiffness ensures that the change in stiffness due to thermal effects is also accounted for. For axial ductility ratio demand, the effective yield displacement, D_y, is calculated using the cross-sectional effective axial stiffness. This axial stiffness is calculated using the material elastic modulus, E_m, and model section thickness, t_m, calibrated in accordance with Section N9.2.3.

6c. Response Determination

One of the following methods can be used to consider dynamic effects of impulsive loads.

(a) The dynamic effects of impulsive loads are considered based on approximation of the wall panel as a single-degree-of-freedom (SDOF) elastic, perfectly plastic system, where the resistance function and limiting ductility are as defined in Section N9.1.6b. System response is determined by either a nonlinear time history analysis, or for well-defined impulse functions (rectangular and triangular pulses), selected from established response charts, such as those in Biggs (1964).

(b) The dynamic effects of impulsive loads are considered based on approximation of the wall panel as a SDOF system with bilinear stiffness. System response is determined by a nonlinear time history analysis. Either the ductility is limited as defined in Section N9.1.6c or the plate principal strain may be limited to 0.05. Application of this approach is described in Johnson et al. (2014).

(c) The dynamic effects of impulsive loads are considered by performing a nonlinear finite element analysis. The plate principal strain is limited to 0.05.

Any rational method can be used to calculate the faceplate thickness required to prevent perforation under projectile impact. Bruhl et al. (2015) have presented the following three-step approach to design an individual SC wall for a specific missile. This method only considers local failure due to missile impact. There may be global responses governing the design. The evaluation procedure is explained in Figure C-A-N9.1.16. The front surface faceplate is conservatively neglected in this analysis. Thus, impact of a projectile (missile) on concrete dislodges a conical concrete plug, which in turn impacts the rear faceplate.

Step 1. The design method involves first selecting a concrete wall thickness, t_c. An existing wall thickness can be used to verify the protection afforded by a given wall. For new designs, the concrete thickness can be obtained from governing design requirements, or 70% of the thickness for an RC wall determined using DOE-STD-3014 (DOE, 2006) or NEI 07-13 (NEI, 2011) is recommended.

Step 2. Next, the residual velocity of the missile after passing through concrete is estimated using the formula in NEI 07-13 (valid for rigid nondeformable missiles with initial velocity less than perforation velocity). The ejected concrete plug is assumed to travel at the same residual velocity as the missile, as the two, together, impact the rear faceplate.

Step 3. The required faceplate thickness, t_p, can then be calculated using the formula presented by Børvik et al. (2009). The corresponding equations for this method are found in Bruhl et al. (2015).

Using the three-step method, graphs can be generated for various missile types or specific wall configurations. Using the procedure outlined in Bruhl et al. (2015), Figure C-A-N9.1.17 has been generated for a flat-nosed, 6-in.-diameter, rigid missile impacting walls of any thickness. Similarly, Figure C-A-N9.1.18 has been generated for the minimum practical SC wall—an interior wall of 12-in.-thick section, t_{sc}, with 0.25-in.-thick faceplates impacted by missiles of various diameters.

For SC walls with 0.015 and 0.050 reinforcement ratios, respectively, Figures C-A-N9.1.17(a) and (b) provide the required concrete wall thickness for an initial missile velocity for a variety of missile weights. Figure C-A-N9.1.18 determines the capacity of the minimum practical SC wall for different missile types. If the specified missile to design against (diameter, weight and initial velocity) falls below the applicable line, the wall will prevent perforation.

An increase of 25% in the faceplate thickness over the value calculated by the empirical methods is necessitated by the scatter in the experimental data. This scatter, which is essentially independent of empirical equations is accounted for by a 25% increase in faceplate thickness based on ASCE Manual and Report Number 58 (ASCE, 1980).

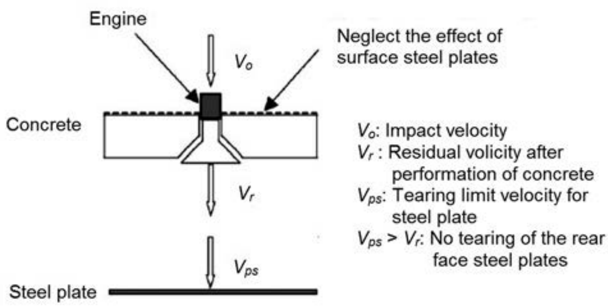
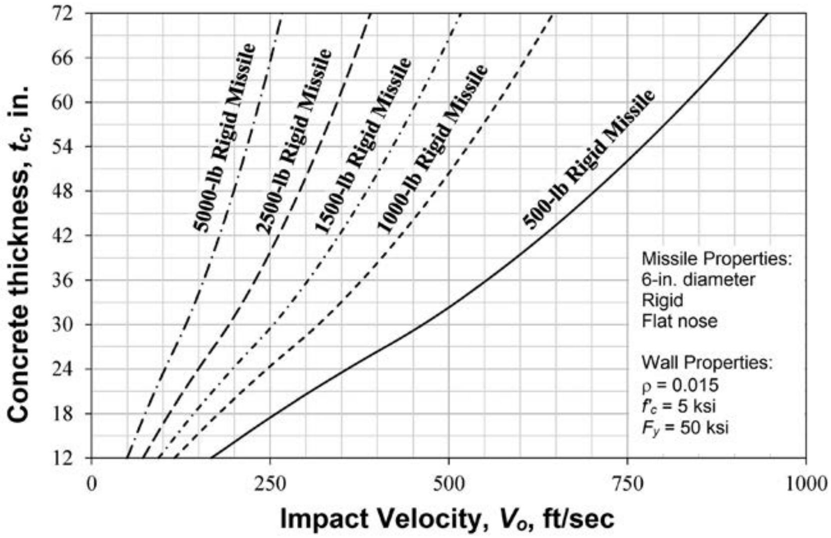
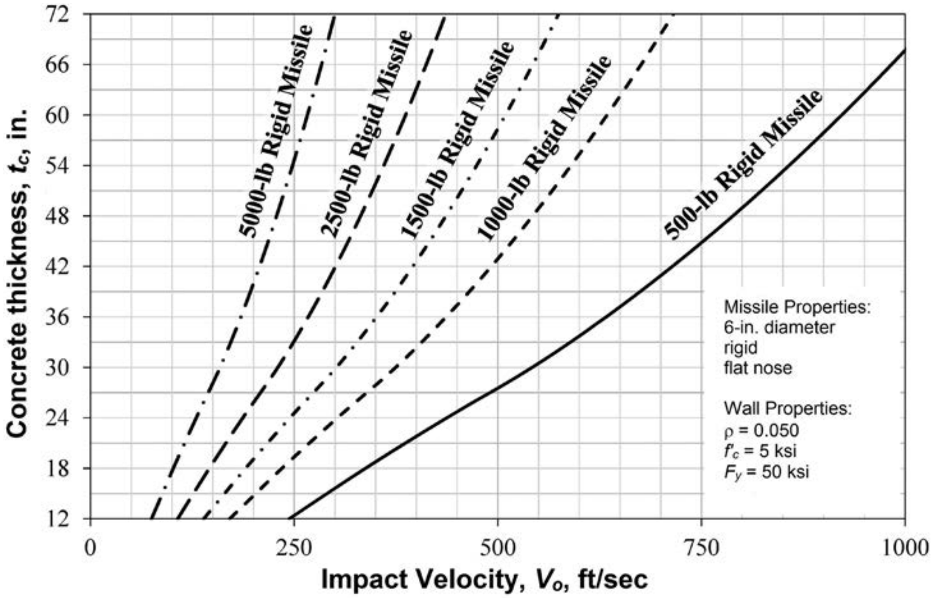


Fig. C-A-N9.1.16. Evaluation procedure for tearing of SC panels against impact (Mizuno et al., 2005).



(a) 6-in.-diameter, flat-nose, rigid missile, 0.015 reinforcement ratio



(b) 6-in.-diameter, flat-nose, rigid missile, 0.050 reinforcement ratio

Fig. C-A-N9.1.17. Required SC wall thickness to prevent perforation.

7. Design and Detailing around Openings

The load redistribution around an opening creates stress concentrations, whose severity depends on factors such as size of the opening, presence/absence of sharp reentrant corners, and type and magnitude of loading. Under severe loading, the faceplate may yield at or near the reentrant corners. However, the area over which yielding occurs and the magnitude of plastic strains remains below the fracture strain limit as long as (1) good detailing practices are used, and (2) the faceplate effective stress due to averaged demands over a small region around the opening is below the yield stress limit (this philosophy is the same as in ASME pressure vessel design).

In addition to the effect on demands, the presence of an opening affects the SC panel section capacity. This happens on two accounts: (1) the region in the vicinity of the opening is not fully effective as an SC section (due to the free edge of steel and concrete at the opening location unless special detailing is provided to achieve a fully developed faceplate at the opening perimeter); and (2) the faceplate has the ability to withstand large plastic strains to help redistribute the demands to regions away from the edges and corners of the opening (e.g., good detailing practices such as avoiding sharp reentrant corners).

The detailing requirements aim at reducing the stress concentration effects and, if desired, achieving a fully developed edge at the opening perimeter. Absent a fully developed edge at the opening perimeter, a fully effective SC panel section will be manifested some distance away from the free edge. The pertinent detailing requirement limits the distance from the free edge to the fully effective SC panel section.

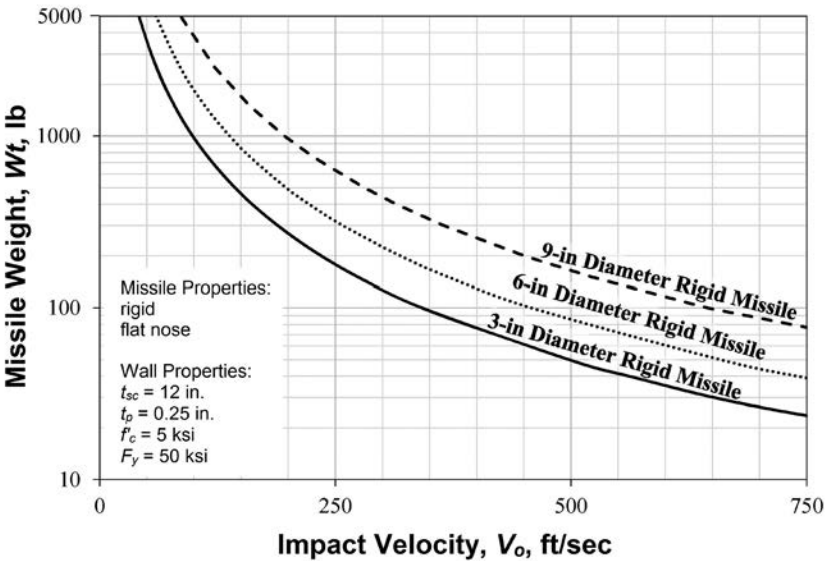


Fig. C-A-N9.1.18. Nondeformable (rigid) missile resistance of minimum SC wall.

Available literature provides data on the effect of small openings on the section strength. This presents the possibility that the effect of small openings can be accounted for by using simple prescriptive rules such that the analytical model need not include small openings. With this in mind, small and large openings are defined based on whether their largest dimension is greater than or less than half times the thickness of the wall. The limit of $t_{sc}/2$ is considered adequately small compared to the evaluation size, $2t_{sc}$, of a panel section for calculating the required strength per Section N9.2.5.

This section provides the modeling, detailing and evaluation criteria to be followed for the SC wall region in the vicinity of small openings and large openings.

7a. Design and Detailing Requirements around Small Openings

To help assure good connection performance, fully developed edges are required for small openings located within the connection region (however, this does not necessarily obviate the need for connection qualification).

(a) Design and detailing for free edge at opening perimeter

Experiments conducted by Japanese researchers (Ozaki et al., 2004) indicate that the maximum decrease in SC panel section capacity is about 15 to 20%.

Based on the test data described in the foregoing, the provisions account for the effect of small openings by conservatively taking a 25% reduction in the capacities of the affected SC panel section(s). In case one panel section encompasses the opening (Opening A in Figure C-A-N9.1.19), the strength of just that panel section needs to be reduced. In case the opening lies in more than one panel section (Opening B in Figure C-A-N9.1.19), the strength of all panel sections that partially include the opening will be reduced by 25%.

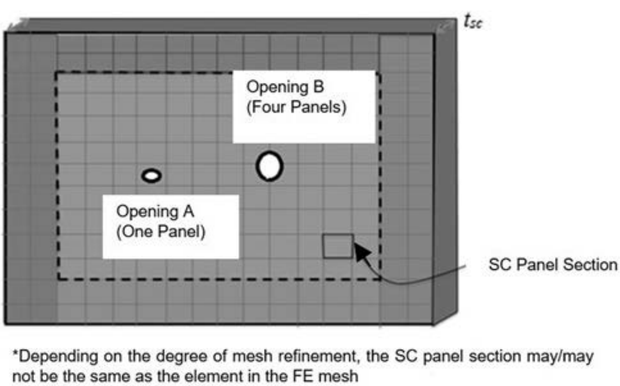


Fig. C-A-N9.1.19. Reduction in strength due to the presence of an opening.

Openings with sharp reentrant corners can still be problematic for the faceplate. The available test data does not clearly address the effect of sharp reentrant corners. Because of these considerations, some provision for corner radii is warranted to avoid the potential for fracture at the sharp corners. The data point for that is derived from AISC Design Guide 2, *Steel and Composite Beams with Web Openings* (Darwin, 1990), for beams with web openings. Figure C-A-N9.1.20 illustrates the radius required to be provided at the reentrant corners. The coping radius, typically twice the thickness, has been limited to four times the thickness to try and further smooth the stress distribution. To help maintain structural integrity against any potential for splitting, a detailing requirement has been provided for locating the first tie within $t_{sc}/4$ from the edge of the opening.

(b) Design and detailing for fully developed edge at opening perimeter

With a fully developed edge at the opening perimeter, the SC panel sections in the vicinity of the opening will be fully effective beginning at the opening edge. A fully developed edge is achieved by providing a welded steel sleeve across the opening. This sleeve has two flange plates welded at its ends to help transfer the faceplate stresses to the sleeve. Normal and tractive stresses at the edge of the faceplate are thus transferred to the sleeve, which in turn transfers them to the concrete infill since it is anchored into concrete using steel anchors. The sleeve and flange plate thickness and yield stress are specified such that faceplate stresses can be adequately transferred to the concrete.

The detailing for the sleeve can be thought of as a cylinder spanning across the SC wall section with annular discs at its two edges. The flange plate is extended a minimum distance of one times the SC wall thickness to provide additional strength in the stress concentration region. As described in the following, the faceplate is welded to either just the flange plates or both the flange plates and the sleeve depending on the thickness of the flange plate:

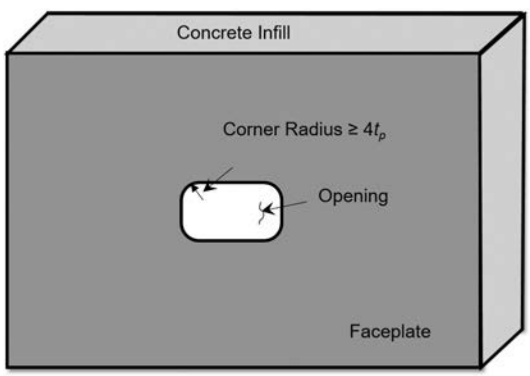


Fig. C-A-N9.1.20. Radius of reentrant corners (elevation view of the SC panel section).

- In the case that the thickness of the flange plate is less than 1.25 times the faceplate thickness, then the faceplate acts as a doubler/reinforcing plate that helps deliver the concentrated stresses to the sleeve (see Figure C-A-N9.1.21).

- If the flange plate is thicker than or equal to 1.25 times the faceplate thickness, it is deemed capable of taking care of the stress concentration effects by itself. Hence, the faceplate need only be welded to the flange plate, which meets up with the sleeve (see Figure C-A-N9.1.22).

No reduction in SC panel section capacities is considered because of exercising either of the above detailing requirements. Furthermore, as in the case of an opening with a free edge, the stress concentration around openings is alleviated by avoiding sharp reentrant corners.

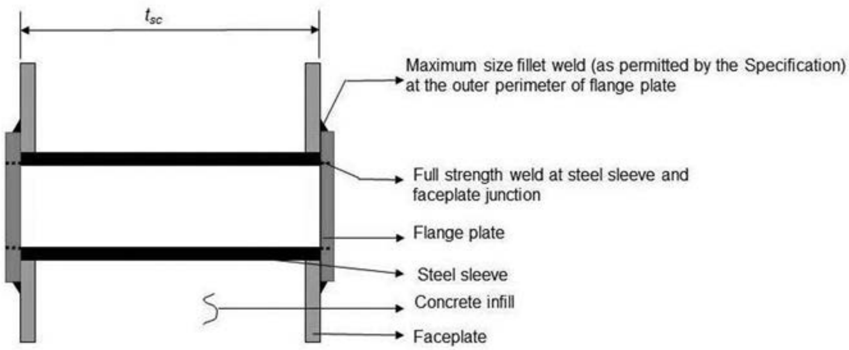


*Fig. C-A-N9.1.21. Small circular opening—detailing illustration for fully developed edge with flange plate thickness ≤ 1.25*t_p.

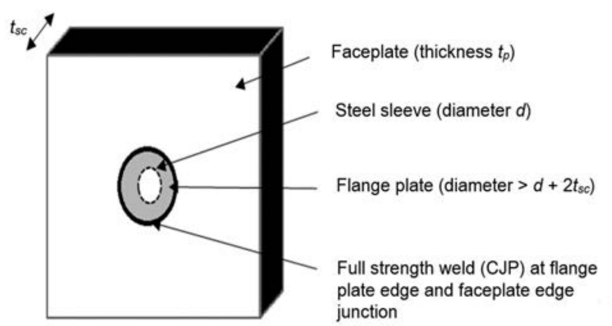


*Fig. C-A-N9.1.22. Small circular opening—detailing illustration for fully developed edge with flange plate thickness ≥ 1.25*t_p.

7b. Design and Detailing Requirements around Large Openings

Compared to the requirements for small openings, a more rigorous set of criteria is followed for large openings.

(a) Design and detailing for free edge at opening perimeter

When detailed as a free edge, the opening is required to be modeled as larger than the physical opening. The composite behavior of a wall section develops fully only after some length (development length). The SC wall in the intervening region cannot attain its full capacity and is, therefore, ignored in the analytical model. According to Section N9.1.4b, the faceplate development length, L_d, has to be no greater than three times t_{sc}, the SC section thickness. Thus, considering a development length of just one times t_{sc}, the as-modeled opening dimension will be two times the section thickness more than the physical opening dimension (Figure C-A-N9.1.23). For example, under this free edge option, a 4-ft-diameter (1.2 m) circular opening in a 4-ft-thick (1.2 m) SC wall will have to be modeled as a 12-ft-diameter (3.7 m) opening, which may severely increase the resulting analysis-based demands for the surrounding SC panel sections (risking the possibility that they will be inadequate unless thicker faceplates are used locally).

Because the region of stress concentration and partial composite action has not been modeled, no reduction in strength needs to be considered for the as-modeled SC wall. As in the case of small openings, stress concentration effects are minimized by providing corner radii at reentrant corners. To help maintain structural integrity against any potential for splitting, a detailing requirement has been provided for locating the first tie within $t_{sc}/4$ from the edge of the opening.

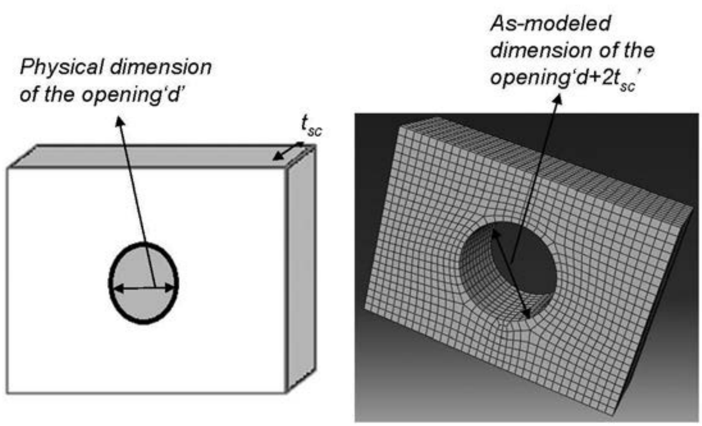


Fig. C-A-N9.1.23. Modeling of large openings with free edge at opening perimeter.

(b) Design and detailing for fully developed edge at opening perimeter

The edge will be fully developed with the same detailing requirements as for small openings. However, the demands need to be obtained by modeling the physical opening.

N9.2. ANALYSIS REQUIREMENTS

1. General Provisions

SC wall structures are modeled using elastic finite elements, as explained earlier in Commentary Section N9.1.2. These finite elements can be thick-shell finite elements or solid finite elements. Finer meshes are used around section penetrations larger than half the wall thickness. The viscous damping ratios for safe shutdown earthquake seismic analysis can be assumed to not exceed 5% and this is based on 1/10th scale tests of the entire containment internal structure consisting of SC modules (Akiyama et al., 1989). However, for custom designs for the operating basis earthquake where in-structure response spectra need to be generated, a damping ratio of 5% is unconservative and lower ratios need to be used (2 to 3%). When using shell elements to model the expanse of the SC walls, it is recommended to use meshes consisting of at least four to six elements along the short direction and six to eight elements along the long direction. These numbers are based on recommendations in ASCE 4 (ASCE, 1998) and will adequately capture local modes of vibration.

Finite elements larger than $2t_{sc}$ are not recommended for the interior regions. Finite elements larger than t_{sc} are not recommended for connection regions and regions around section penetrations. These element size limits are recommended based on the design capacity equations that are deemed appropriate up to $2t_{sc} \times 2t_{sc}$, i.e., the equations do not apply to the whole wall.

2. Effective Stiffness for Analysis

(a) Effective flexural stiffness for analysis of SC walls

Experimental studies by Booth et al. (2007) and Varma et al. (2009, 2011a) indicate that the uncracked composite flexural stiffness is generally not manifest in SC walls. This is due to effects of locked-in shrinkage strains in the concrete core, partial composite action of the section, and reduced bond parameter due to discrete steel anchor locations.

The cracked transformed flexural stiffness of the SC wall for a wide range of parameters can be expressed using the stress, strain and force block in Figure C-A-N9.2.1, where n is the concrete-to-steel modular ratio, E_c/E_s, e_c is the top plate strain, c is the distance to the neutral axis, and strain compatibility between extreme concrete fibers and faceplates is assumed. The faceplate thickness is neglected while plotting the strain diagram. Also, cubic terms of t_p have been ignored when calculating the cracked transformed stiffness (Equation C-A-N9-3a).

Equilibrium of forces in Figure C-A-N9.2.1 results in Equation C-A-N9-1a for neutral axis depth, wherein ρ' is the stiffness normalized reinforcement ratio (Equation C-A-N9-2).

$$\frac{c}{t_{sc}} = \sqrt{(\rho')^2 + \rho'} - \rho' \qquad \text{(C-A-N9-1a)}$$

$$\rho' = \frac{2t_p E_s}{t_{sc} E_c} \qquad \text{(C-A-N9-2)}$$

The corresponding flexural stiffness, $(EI)_{cr\text{-}tr}$, per unit width can then be calculated as follows.

$$(EI)_{cr\text{-}tr} = E_s \left\{ 12 t_p t_{sc}^2 \left[1 + 2\left(\frac{c}{t_{sc}}\right)^2 - 2\frac{c}{t_{sc}} - \frac{t_p}{t_{sc}} \right] + \frac{4t_{sc}^3}{n}\left(\frac{c - t_p}{t_{sc}}\right)^3 \right\}$$

(C-A-N9-3a)

However, Varma et al. (2011a) calibrated this equation to the simpler form given by Equation C-A-N9-4:

$$(EI)_{cr\text{-}tr} = E_s I_s + c_2 E_c I_c \qquad \text{(C-A-N9-4)}$$

where

$c_2 = 0.48\rho' + 0.10$ (C-A-N9-5)

Figure C-A-N9.2.2 shows the calibration of c_2 as a function of ρ'.

The expressions can also be derived considering the faceplate thickness. The corresponding expressions for c/t_{sc} and $(EI)_{cr\text{-}tr}$ are given in Equations C-A-N9-1b and C-A-N9-3b. It is observed that the values using these equations match closely with those obtained using the simplified method (Equations C-A-N9-1a and C-A-N9-3a).

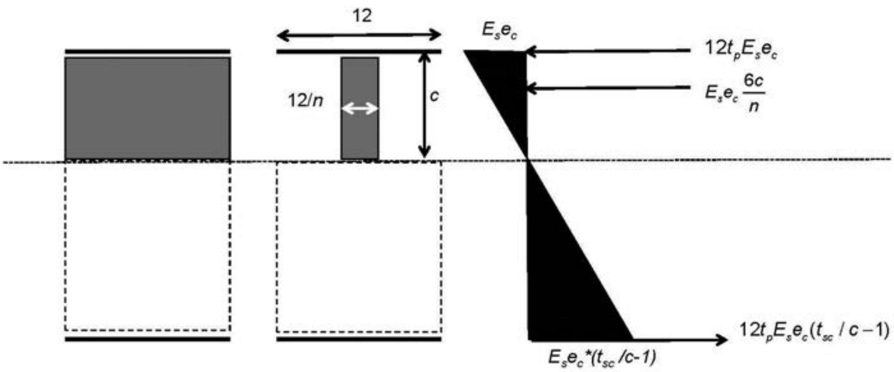


Fig. C-A-N9.2.1. Flexural stiffness of cracked-transformed section of SC walls.

$$\frac{c}{t_{sc}} = \sqrt{(\rho')^2 + \rho' - \rho\rho'} - \rho' + \frac{\rho}{2} \tag{C-A-N9-1b}$$

$$(EI)_{cr\text{-}tr} = E_s\left\{8t_p^3 + 12t_p t_{sc}^{\,2}\left[1 + 2\left(\frac{c}{t_{sc}}\right)^2 - 2\frac{c}{t_{sc}} - \frac{t_p}{t_{sc}}\right] + \frac{4t_{sc}^{\,3}}{n}\left(\frac{c - t_p}{t_{sc}}\right)^3\right\} \tag{C-A-N9-3b}$$

The Booth and Varma studies have further shown that ambient thermal loading conditions produce linear thermal gradients, which develop gradually over time. As a result, there is little to no additional concrete cracking due to ambient thermal loading and the cracked-transformed section flexural stiffness applies. However, accident thermal loading increases the faceplate temperature rapidly, while the concrete temperature lags behind. In addition, a nonlinear temperature gradient develops through the composite cross section because of the significantly lower thermal conductivity of concrete and this gradient results in cracking of the concrete due to its low tensile stress, f_t'.

The flexural stiffness recommendation accounts for the potential cracking of the concrete due to the accident thermal gradient through the composite section. It considers temperature increases greater than 150°F (83°C) on the faceplates to result in full (through-section) concrete cracking, i.e., the flexural stiffness will be equal to that of the steel, E_sI_s, alone. For faceplate surface temperature change from 0 to 150°F (–21 to 66°C), the cracked transformed flexural stiffness, $E_sI_s + c_2E_cI_c$, is linearly reduced until it equals the steel section stiffness, E_sI_s, which is the minimum effective flexural stiffness. ΔT_{savg} is calculated by taking the average of the maximum surface temperature increases on the two faceplates (ΔT_{s1}^{max} and ΔT_{s2}^{max}) due to accident thermal conditions.

$$\Delta T_{savg} = \frac{\left(\Delta T_{s1}^{max} + \Delta T_{s2}^{max}\right)}{2} \tag{C-A-N9-6}$$

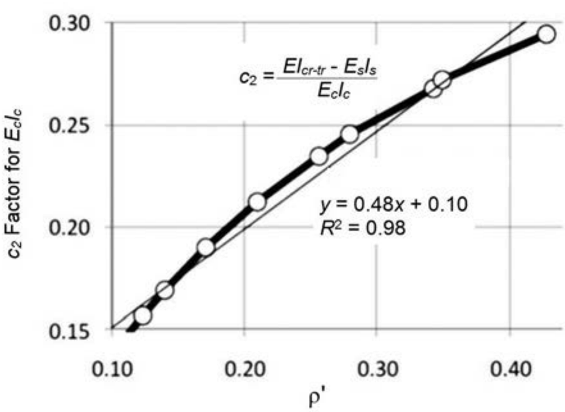


Fig. C-A-N9.2.2. Calibration of c_2 *versus* ρ' *(Varma et al., 2011a).*

(b) Effective in-plane shear stiffness of SC walls for all load combinations that do not involve accident thermal loads

The in-plane shear behavior of SC walls is governed by the plane-stress behavior of the faceplates and orthotropic cracked behavior of the concrete infill. Ozaki et al. (2004) and Varma et al. (2011b) have developed a trilinear shear force-shear strain model for SC walls with reinforcement ratios, ρ, from 0.015 to 0.050. This model is discussed in Commentary Section N9.3.4.

According to this mechanics-based model, composite uncracked behavior of the SC wall occurs when the in-plane shear force is less than or equal to the cracking threshold, S_{cr}, given by:

$$S_{cr} = \left(\frac{0.126\sqrt{f_c'}}{G_c} - \varepsilon_{sh} \right)\left(G_c A_c + G A_s\right) \quad \text{(C-A-N9-7)}$$

$$S_{cr} = \left(\frac{10.5\sqrt{f_c'}}{G_c} - \varepsilon_{sh} \right)\left(G_c A_c + G A_s\right) \quad \text{(C-A-N9-7M)}$$

Figure C-A-N9.2.3 shows a plot of experimental versus calculated values of cracking strength by Varma et al. (2014). The cracking strength, S_{cr}, is calculated assuming the shrinkage strain, ε_{sh}, to be $0.063\sqrt{f_c'}/G_c$ (S.I.: $5.25\sqrt{f_c'}/G_c$). The pre-cracking shear stiffness can be estimated as the composite shear stiffness, $GA_s + G_cA_c$. It is important to understand that the composite action between the faceplates and the concrete infill (through the steel anchors, ties, etc.) is discrete and not perfect.

After cracking, the tangent stiffness is governed by the cracked orthotropic behavior of concrete acting compositely with faceplates that are in a state of plane stress. The tangent stiffness, K_{xy}^{cr}, can be estimated as $K_s + K_{sc}$, where

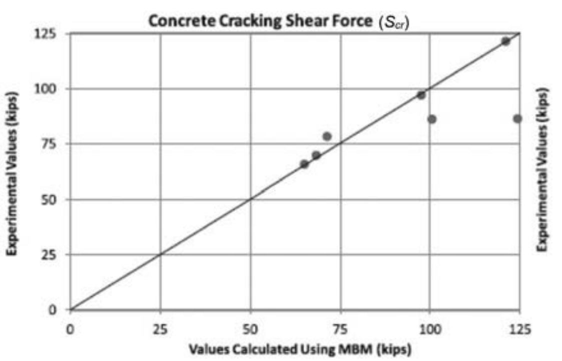


Fig. C-A-N9.2.3. Experimental versus calculated values of cracking strength (Varma et al., 2014).

$$K_s = G2t_p \tag{C-A-N9-8}$$

$$K_{sc} = \frac{1}{\dfrac{4}{0.7E_c t_c} + \dfrac{2(1-\nu)}{E_s 2t_p}} \tag{C-A-N9-9}$$

where

K_s = contribution of faceplates to in-plane shear stiffness

K_{sc} = contribution of cracked orthotropic concrete to in-plane shear stiffness

ν = Poisson's ratio of steel

However, under seismic loading, the cyclic behavior of SC walls is governed by secant stiffness, K_{xy}^{sec}, not tangent stiffness. The secant stiffness can be estimated as a function of the applied shear force, S_{rxy}. Figure C-A-N9.2.4 illustrates the variation of normalized secant stiffness with normalized in-plane shear force for different values of the strength-adjusted reinforcement ratio, $\bar{\rho}$. The secant stiffness, K_{xy}^{sec}, is normalized with respect to the uncracked stiffness, K_{xy}^{uncr}, and the applied shear force, S_{rxy}, is normalized with respect to the nominal in-plane shear strength, V_{ni}, as calculated in Section N9.3.4. It is observed in Figure C-A-N9.2.4 that the secant stiffness drops exponentially after occurrence of cracking and reaches the cracked stiffness, K_{xy}^{cr}, asymptotically.

Considering this variation in the secant stiffness, Varma et al. (2011a) developed a simple model for estimating the secant stiffness of SC walls (Figure C-A-N9.2.5). The equations for in-plane shear stiffness of SC walls are based on this model. For in-plane shear force values, S_{rxy}, less than the cracking threshold, S_{cr},

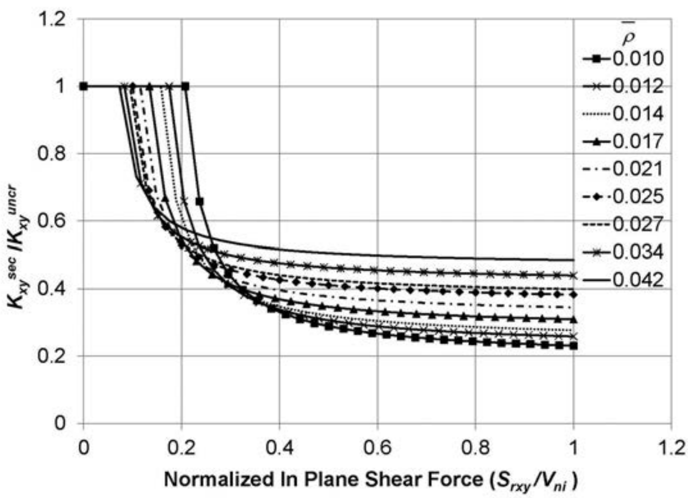


Fig. C-A-N9.2.4. Variation of secant stiffness of SC walls (Varma et al., 2011a).

the effective secant stiffness, K_{xy}^{sec}, is the uncracked stiffness of the section. For S_{rxy} values greater than twice the cracking threshold, the effective stiffness is the post-cracking shear stiffness. Between S_{cr} and $2S_{cr}$, S_{rxy} is determined by linear interpolation.

The use of stainless steel plates does not change the in-plane shear behavior (stiffness and strength) of SC walls. The concrete infill is still the major contributor to the in-plane shear stiffness before and after cracking. The contribution of the stainless steel faceplates can be accounted for appropriately by using the value of shear modulus, G, from the Symbols list. Additionally, the in-plane shear strength Equation A-N9-20 will be slightly conservative for stainless steel plates due to its lower elastic modulus and early onset of strain hardening.

(c) Effective in-plane shear stiffness, GA_{eff}, for all loading combinations involving accident thermal conditions

The in-plane shear stiffness of SC walls after accident thermal loading was evaluated experimentally by researchers in Japan (Ozaki et al., 2000). As discussed in Varma et al. (2011a), nonlinear (parabolic) thermal gradients develop through the concrete section due to the loading. This gradient induces concrete cracking in two orthogonal directions due to the expansion of faceplates and the low cracking threshold of the concrete. The accident thermal loading eliminates the uncracked shear force-strain behavior. Thus, the in-plane shear stiffness of SC walls after accident thermal loading can be estimated as the post-cracking shear stiffness of the composite section, $K_s + K_c$, i.e.,

$$K_{xy}^{cr} = 0.5\left(\bar{\rho}^{\,-0.42}\right)GA_s \qquad \text{(C-A-N9-10)}$$

These orthogonal cracks due to thermal loading do not reduce the in-plane shear strength of SC wall panels significantly.

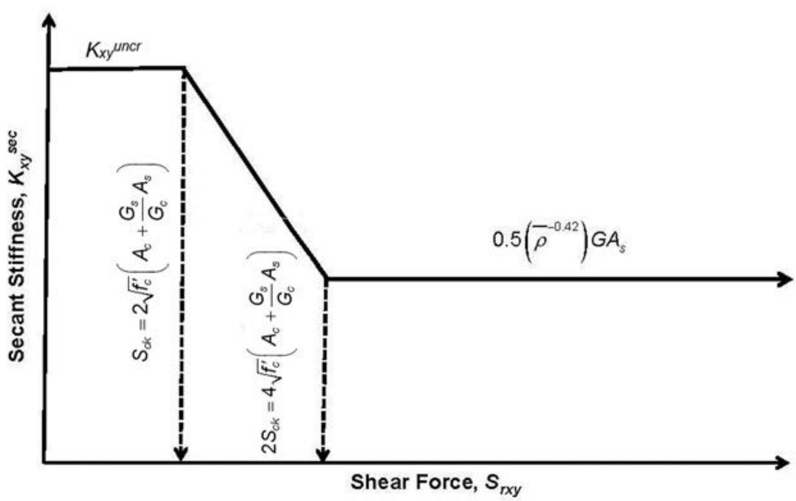


Fig. C-A-N9.2.5. A simple model for secant stiffness with no accident thermal loading (Varma et al., 2011a).

3. Geometric and Material Properties for Finite Element Analysis

An elastic finite element model of the composite SC section is required to be developed using a single material. As mentioned earlier, this model is used for dynamic soil structure interaction and subsequent analysis. For this single material elastic model, the following steps are implemented to determine the material properties:

(a) Match the Poisson's ratio, thermal expansion coefficient, and thermal conductivity of the material to those of concrete because these parameters will govern the thermally induced displacements of the structure.

(b) Calibrate the model section thickness and material elastic modulus so that the effective stiffness of the model match those of the physical SC wall section.

(c) Calibrate the material density to match the mass of the model with that of the physical section.

(d) Calibrate the material specific heat to match that of the concrete. This will allow transient heat transfer analysis to be accurately conducted using the elastic, single material, finite element model.

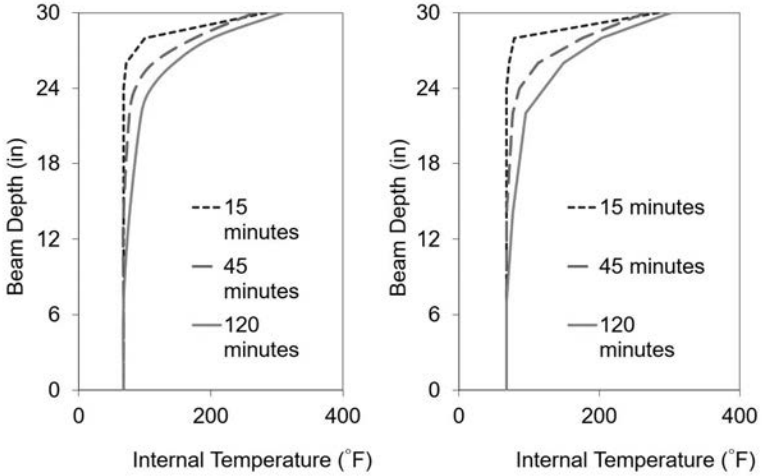
4. Analyses Involving Accident Thermal Conditions

Booth et al. (2007) and Varma et al. (2009) performed experimental and analytical studies to evaluate the effect of thermal loads (ambient and accident) on the behavior of SC walls. It was concluded from the Booth study that ambient stiffness of the composite walls can be predicted using cracked transformed section properties. Upon applying accidental thermal loads, a nonlinear thermal gradient develops across the concrete cross section, causing the concrete to crack in tension (see Figure C-A-N9.2.6).

Figure C-A-N9.2.6 compares the experimental temperatures and thermal gradients with those obtained from a fiber model. This fiber model was then used to predict the moment-curvature, M-ϕ, response of the SC walls for the design thermal loading. Figure C-A-N9.2.7 presents the M-ϕ responses predicted for the specimen. The figure shows that the thermal gradient shifts the diagram to the left with nonzero thermal curvature, ϕ_{th}, at zero moment and nonzero thermal moment, M_{th}, at zero curvature. Figure C-A-N9.2.8 shows that the thermal moment, M_{th}, can be related to the thermal curvature, ϕ_{th}, using the fully cracked section stiffness.

The stiffness of the SC wall subjected to ΔT_{savg} greater than or equal to 150°F (83°C) can be predicted using fully cracked (steel only) section properties. Based on the preceding results, Varma et al. (2009) developed the simple equations given in the Nuclear Specification to predict the effects of combined thermal and mechanical loading in locations away from supports. These equations do not apply at supports that may be fully restrained from expansion.

Temperature dependent properties for steel are not required for temperatures up to 400°F (200°C). For temperatures greater than 400°F (200°C), temperature dependent properties from Appendix N4 are recommended for use.



(a) Analytically determined thermal gradient

(b) Experimentally determined thermal gradient (fiber model)

Fig. C-A-N9.2.6. Comparison of analytically and experimentally determined thermal gradients (Varma et al., 2009).

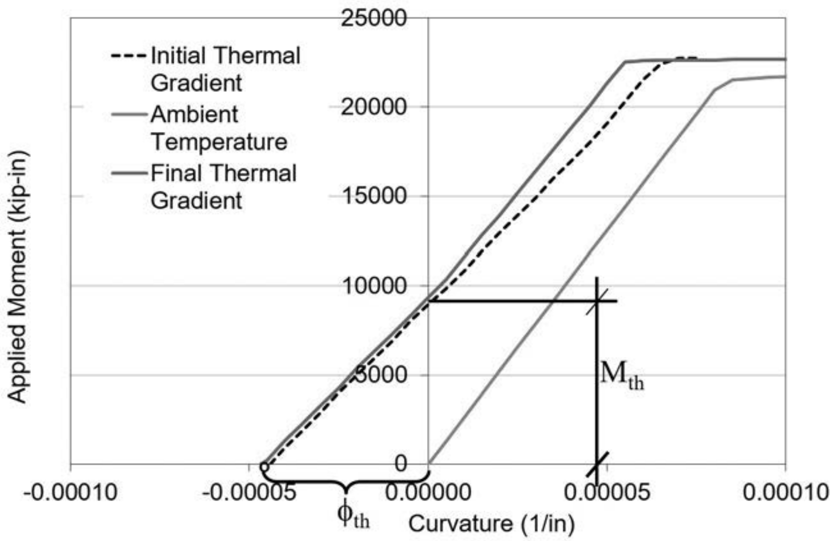


Fig. C-A-N9.2.7. Comparison of fiber model moment curvature to transformed cracked and fully cracked moment of inertia (Varma et al., 2009).

5. Determination of Required Strengths

Averaging and design assessment for interior regions is done over $2t_{sc}$ by $2t_{sc}$ panel sections, because the size represents reasonable but not extensive yielding (first onset of significant inelastic deformation at the safe shutdown earthquake level). While the development length, L_d, is limited to three times the section thickness, $3t_{sc}$, a lower value for averaging has been used because $3t_{sc}$ is deemed to be very large considering typical SC wall thicknesses, e.g., for a 4-ft-thick (1.2 m) SC wall, keeping panel section dimensions at $3t_{sc}$ would result in 12 ft by 12 ft (3.7 m by 3.7 m) panel sections. This size may result in very few panel sections per wall leading to less accurate determination of demands for the SC walls. Averaging in connection regions and regions around openings has also been limited to t_{sc}, compared to the L_d value of $2t_{sc}$, for the same reasons.

Also, $3t_{sc}$ is a notional value for the development length. In most cases, the faceplates of SC walls will be directly welded (to steel baseplates or other faceplates), which will develop them immediately at the weld location itself. Developing the faceplate yield strength over the panel sections would not be an issue in most cases. The sizing recommendations for panel sections are illustrated in Figure C-A-N9.2.9.

N9.3. DESIGN OF SC WALLS

Concrete contribution to the tensile strength of the section has not been considered. Neglecting concrete tensile capacity is appropriate for SC sections since they experience a higher degree of cracking due to curing shrinkage than typically observed in reinforced concrete sections. This is due to locked-in tensile stresses in the SC concrete core that result from restraint of curing shrinkage by the faceplates, and also the discrete nature of the bond between the reinforcing steel and the concrete core. The steel ribs are provided primarily to increase faceplate stiffness and strength to handle rigging and construction loads (e.g., wet concrete pressure). Therefore, the contribution of the steel ribs to available strength is neglected.

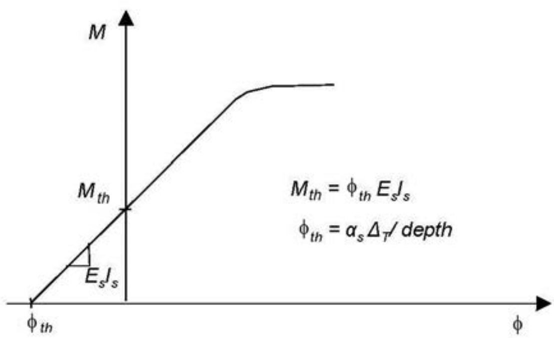


Fig. C-A-N9.2.8. Relationship between moment and thermal gradient.

1. Uniaxial Tensile Strength

The reduction in available tensile strength of the SC panel sections due to holes in the faceplates is taken care of by avoiding tensile rupture in the faceplates.

2. Compressive Strength

The SC wall panel sections are designed by calculating their available axial compressive strength on a per foot basis. The calculation uses the clear length of the wall along the direction of loading and an effective SC stiffness per unit width for buckling evaluation, which is based on EI_{eff} of filled composite columns in *Specification* Chapter I. The equation for EI_{eff} for filled composite columns has been simplified conservatively to $E_sI_s + 0.60E_cI_c$. The more accurate equation in *Specification* Chapter I, which is a function of the reinforcement ratio, can also be used. Additionally, the effective length factor, K, has been conservatively considered equal to 1.

Equation A-N9-16 gives the nominal compressive strength for SC wall panel sections with nonslender faceplates at ambient temperatures. Varma et al. (2013) used benchmarked finite element models to analytically study the impact of elevated temperatures on the compressive strength of an SC wall.

Figure C-A-N9.3.1 shows the analysis results for different temperature magnitudes. The compressive strength of the analytical models has been normalized with respect to the available strength calculated using Equation A-N9-16. The equation becomes slightly unconservative for temperatures above 482°F (250°C). The figure also indicates that the duration of heating (30 minutes or three hours) does not affect the compressive strength of SC walls. Therefore, Equation A-N9-16 is recommended for calculating the available compressive strength of SC wall panel sections subjected to accident thermal loading causing surface temperatures up to 300°F (150°C).

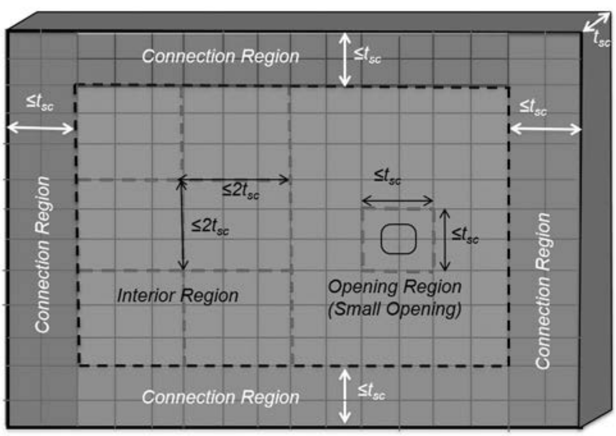
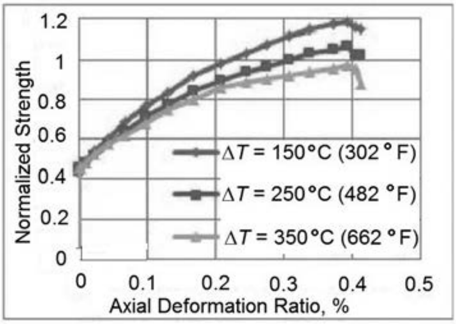


Fig. C-A-N9.2.9. Panel section sizing for averaging the design demands.

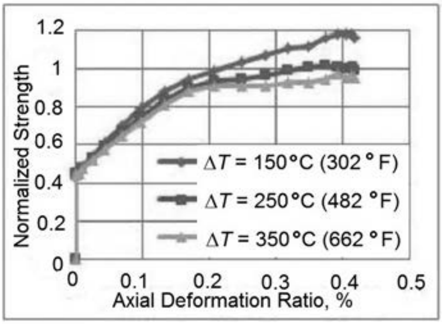
3. Out-of-Plane Flexural Strength

The nominal flexural strength, M_n, can also be calculated using the reinforced concrete principles mentioned in Section 10.2 of ACI 349-13 or ACI 349M-13 (ACI, 2013). The design assumptions and limitations for determining flexural capacity of concrete members listed in the section can be applied to SC walls with slight modifications accounting for the differences with reinforce concrete design, particularly having the faceplates on the exterior faces (Sener et al., 2015).

SC design is inherently similar to that of doubly reinforced concrete beams. Therefore, the faceplate in compression will not yield before the concrete in compression is fully crushed, or the neutral axis is located under the compression faceplate. This limits the strain in the extreme fiber of the concrete in compression to the steel yield strain. Concrete stress variation can be approximately assumed to be linear up to strain equal to the yield strain of typically used faceplates (about 2,000μ). Assuming a triangular stress variation in concrete below this strain level and transforming the compression faceplate to an equivalent concrete block, the nominal flexural strength, M_n, can be calculated by summing moments about the centroid of the transformed block (stress in the transformed concrete block is assumed equal to the smaller of f_c' or F_y/n).



(a) s/t$_p$ *= 10, time = 30 minutes*



(b) s/t$_p$ *= 20, time = 30 minutes*

Fig. C-A-N9.3.1. Load displacement curves: temperature magnitude as parameter (Varma et al., 2013).

Ignoring the contribution of steel ribs, Equation C-A-N9-11 gives the resultant expression, where c_c is the depth of the triangular concrete compressive block.

$$M_n = \left[A_s^F F_y \left(t_{sc} - t_p \right) - \frac{1}{2} flc_c \left(\frac{c_c}{3} + \frac{t_p}{2} \right) \right] \quad \text{(C-A-N9-11)}$$

where

A_s^F = cross-sectional area of the faceplate in tension due to flexure per unit width, in.^{2}/ft (mm^2/m)

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

$$c_c = 2t_p \left(\frac{F_y}{f_c'} - n \right) \geq 0 \quad \text{(C-A-N9-12)}$$

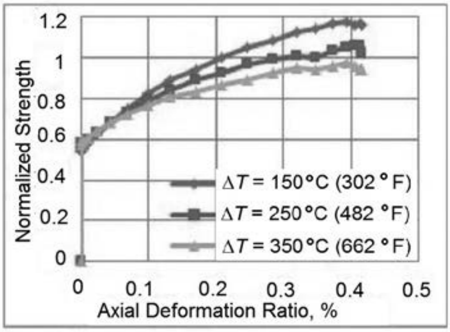
f = F_y/n or f_c', whichever is less

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

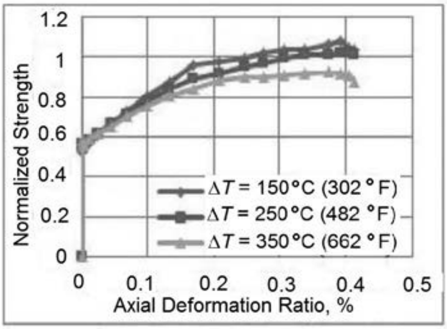
t_p = thickness of the faceplate, in. (mm)

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

n = modular ratio (E_s/E_c)



(c) s/t_p *= 10, time = 3 hours*



(d) s/t_p *= 20, time = 3 hours*

Fig. C-A-N9.3.1 (continued). Load displacement curves: temperature magnitude as parameter (Varma et al., 2013).

Sener et al. (2015) compared the nominal flexural strength values obtained using Equation C-A-N9-11 (modified to include the contribution of the steel ribs) with flexural strength data obtained from experimental studies by Japanese (Ozaki et al., 2001), South Korean (Hong et al., 2009), and U.S. (Varma et al., 2011c) researchers. Figure C-A-N9.3.2 plots the experimental out-of-plane strength data normalized with modified Equation C-A-N9-11. As shown, the flexural strength equation is conservative for the majority of the specimen capacities. It is observed that there is no clear trend between the flexural strength and section depth.

4. In-Plane Shear Strength

The in-plane shear behavior of the SC walls is governed by the plane stress behavior of the faceplates and the orthotropic elastic behavior of concrete cracked in principal tension. Ozaki et al. (2004) and Varma et al. (2011b) developed the fundamental in-plane behavior mechanics-based model for SC walls. The in-plane shear strength of SC walls can be estimated as the trilinear shear force-strain curve shown in Figure C-A-N9.3.3. The first part of the curve is before the concrete cracks. The second part is after the concrete cracking but before the faceplate yielding. The third part of the curve corresponds to the onset of faceplate Von Mises yielding. The shear force corresponding to the onset point is the yield shear strength, S_{rxy}^{Y}, of the section, given by

$$V_{ni} = S_{rxy}^{Y} = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}}\left(2t_p F_y\right) \tag{C-A-N9-13}$$

where

$$K_s = G2t_p \tag{C-A-N9-8}$$

$$K_{sc} = \frac{1}{\dfrac{4}{0.7E_c t_c} + \dfrac{2(1-\nu)}{E_s 2t_p}} \tag{C-A-N9-9}$$

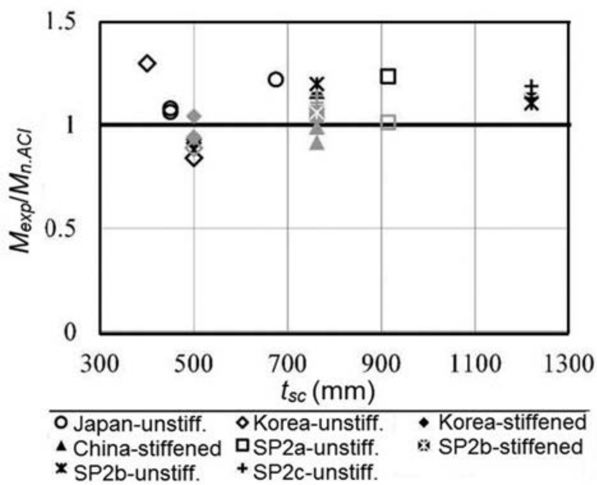


Fig. C-A-N9.3.2. Comparison of experimental flexural strength data with strength using modified Equation C-A-N9-11 (Sener et al., 2015).

This equation was calibrated to the simplified Equation A-N9-20:

$$V_{ni} = \kappa A_s F_y \tag{A-N9-20}$$

where

$$\kappa = 1.11 - 5.16\bar{\rho}$$

$$\bar{\rho} = \frac{A_s F_y}{31.6 A_c \sqrt{f_c'}} \tag{A-N9-13}$$

$$= \frac{A_s F_y}{83 A_c \sqrt{f_c'}} \tag{A-N9-13M}$$

The calibration of κ is shown in Figure C-A-N9.3.4. The values of $\bar{\rho}$ are between 0.01 and 0.04 for nuclear structures. Thus, the in-plane shear behavior is a function of $\bar{\rho}$. The calculation of shear stiffnesses for the three parts is discussed in Commentary Section N9.2.2(b). Varma et al. (2014) compared the in-plane shear strength of the specimen predicted by the mechanics-based model with the experimental results. Figure C-A-N9.3.5 shows that the calculated and experimental values match closely, with the calculated mechanics-based model values being conservative.

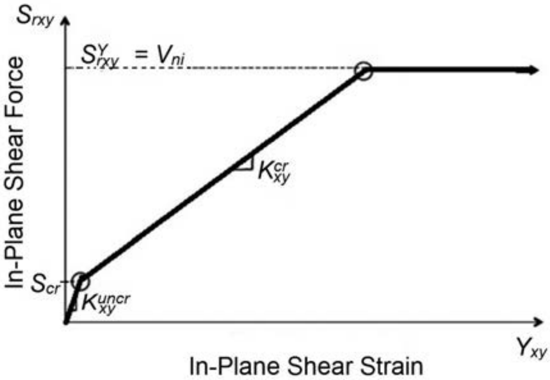


Fig. C-A-N9.3.3. In-plane shear strain curve (Varma et al., 2011b).

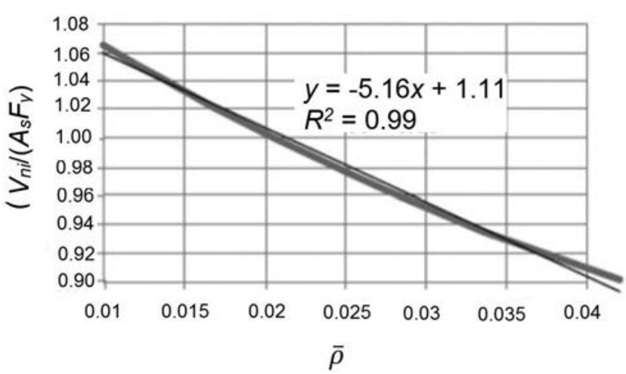


Fig. C-A-N9.3.4. Calibration of $\bar{\rho}$ versus (V_{ni}/A_sF_y)

5. Out-of-Plane Shear Strength

The out-of-plane shear behavior of SC walls is similar to that of reinforced concrete walls with some differences associated with crack spacing, width, etc., due to the more discrete nature of the bond (via steel anchors) in SC walls. Japanese (Ozaki et al., 2001), South Korean (Hong et al., 2009), and U.S. (Varma et al., 2011c) researchers have done extensive experiments to study the out-of-plane behavior of SC sections. Sener and Varma (2014) have compared the shear strengths obtained from this experimental database with the ACI 349-13 or ACI 349M-13 (ACI, 2013) shear strength equations.

Figure C-A-N9.3.6(a) shows the plot of shear strengths obtained from the specimen, normalized with the strength from ACI provisions, and varying with shear span-to-depth ratios. There is a clear trend in the plot where the increase in shear span-to-depth ratio results in a decrease in the strength of both reinforced and unreinforced specimens. The lower bound shear strength is observed to be occurring when the shear span-to-depth ratio is in the approximate 3.0 to 3.5 range. The same normalized shear strength is shown, this time with section depth as the variable, in Figure C-A-N9.3.6(b). Similar variation is seen in the Figure, i.e., with the increase in section depth the shear strength is reduced for both unreinforced and reinforced specimens. This phenomenon is due to size effects in concrete and shows the importance of project-specific large-scale out-of-plane shear tests.

Section N9.1.5a requires classification of the shear reinforcement (ties) as yielding or nonyielding. Currently, both types of shear reinforcement are permitted. The resistance and safety factors, ϕ_{vo} and Ω_{vo}, respectively, for out-of-plane shear reflect the

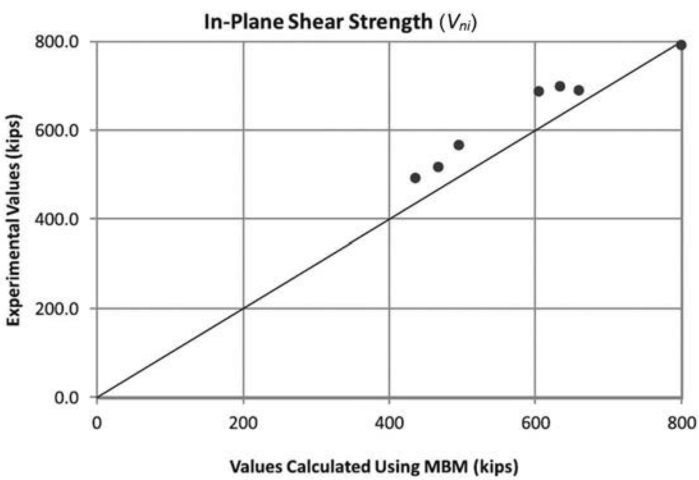
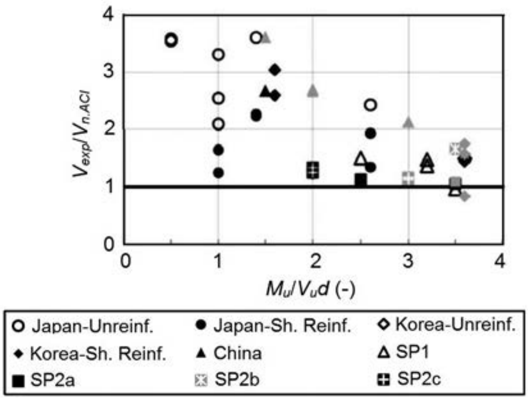


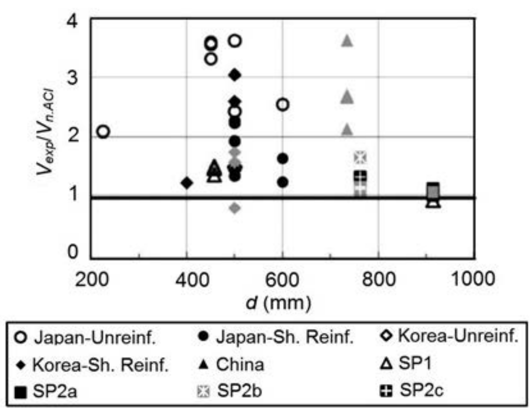
Fig. C-A-N9.3.5. Experimental versus calculated values of in-plane shear strength (Varma et al., 2014).

nonductile nature of the failure mode. The nominal shear strength, V_{no}, is given as the summation of two parts, where V_{conc} is the out-of-plane shear strength contribution of the concrete and V_s is the out-of-plane shear strength contribution from the shear reinforcement (ties).

The shear reinforcement contribution is based on the well-known mechanism of a shear or flexure-shear crack passing through several yielding or nonyielding-type shear reinforcement ties, and engaging them in axial tension. The classification of the shear reinforcement (or ties) as yielding or nonyielding and the determination of its available axial tensile strength are important for this calculation. The concrete contribution has been conservatively taken as $0.05\sqrt{f_c'}$ in ksi ($4.0\sqrt{f_c'}$ in MPa).



(a) Variation with shear span-to-depth ratio



(b) Variation with section depth

Fig. C-A-N9.3.6. Comparison of experimental out-of-plane shear strength data with strength using ACI equations (Sener and Varma, 2014).

When the spacing of the yielding shear reinforcement is greater than half the section thickness, the maximum out-of-plane shear strength is limited to the larger of (i) the concrete shear strength contribution, or (ii) the steel contribution alone. This is based on the ability of the SC beam to develop an internal truss mechanism for equilibrium. The strength of this truss mechanism is limited to that of the tie (shear reinforcement). The concrete and steel contributions cannot be added for shear reinforcement spacing greater than half the section thickness because the shear or flexural-shear crack may not pass through more than one shear reinforcement tie.

For nonyielding shear reinforcement, spaced no greater than half the section thickness, it is feasible that the concrete shear or flexure shear crack will activate all the individual shear reinforcements that it will pass through. However, it is unclear whether these individual shear reinforcements will be able to develop their individual axial available strength before one of them (the one with the largest axial force) fails in a nonductile manner. Hence, the shear reinforcement contribution has been reduced by half.

Requirements for nonyielding shear reinforcement with spacing greater than half the wall thickness are the same as those for yielding shear reinforcement spaced at more than half the wall thickness, with the reasoning being the same.

6. Strength Under Combined Forces

6a. Out-of-Plane Shear Forces

The out-of-plane shear demands, V_{rx} and V_{ry}, both rely on using the same shear reinforcement for their steel contributions, V_s. Both V_{rx} and V_{ry} subject the shear reinforcement to axial tension demand after the concrete cracks and its contribution, $V_{c\ conc}$, in respective directions is exceeded. Therefore, a linear interaction is assumed and the shear reinforcement is checked to ensure that it is not overstressed (yielded) by the combinations of demands.

In the first part of the linear interaction equation, the numerators are the portion of the demands greater than the corresponding concrete contributions, $V_{c\ conc}$. The denominators are the contributions of the shear reinforcement, V_s. The second term in the interaction equation is due to the participation of ties and steel anchors in resisting interfacial shear force. It uses the vector sum of the shears, V_{rx} and V_{ry}, and is obtained by manipulation of Equation A-N9-4.

The weighted average of shear strength contributions of ties and steel anchors, Q_{cv}^{avg}, can be calculated as follows.

$$Q_{cv}^{avg} = \frac{n_{et} Q_{cv}^{tie} + n_{es} Q_{cv}}{n_{et} + n_{es}} \qquad \text{(C-A-N9-14)}$$

where

Q_{cv}^{tie} = available interfacial shear strength of the tie bars, per Section N9.1.4a, kips (N)

n_{et} = effective number of ties contributing to a unit cell

n_{es} = effective number of steel anchors contributing to a unit cell

The unit cell is the quadrilateral region between four ties. It is illustrated in Figure C-A-N9.3.7 for an SC wall of thickness 36 in. (900 mm), with ties spaced at 36 in. (900 mm) and steel anchors spaced at 9 in. (225 mm). With a quarter of the tie at each corner contributing to the unit cell, n_{et} for the case will be 1. The steel anchors inside the cell will contribute completely, but those on the edges will have 50% contribution. Hence, for this example, the effective number of steel anchors contributing to the unit cell, n_{es}, will be $\left[(1)(9)+(0.5)(12)\right]=15$.

When the required out-of-plane shear strength in a given direction (i.e., V_{rx} or V_{ry}) is less than the concrete contribution, the shear reinforcement is not subjected to that demand (i.e., no forces will be incurred in the shear reinforcement because the concrete strength is adequate). Hence, there will be no interaction of out-of-plane shear demands in that case. For shear reinforcement spaced greater than half the section thickness, the available strength will be equal to the greater of the shear reinforcement (steel) and the concrete contributions. In the case of the steel contribution being more, the concrete contribution term in the equation will go to zero. If the concrete contribution is more, the concrete infill will be subject to two-way shear (punching shear), which will be resisted by the unit perimeter of the panel section.

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

The design adequacy of SC panel sections for the combined in-plane forces (S_{rx}, S_{ry}, S_{rxy}) and out-of-plane moments (M_{rx}, M_{ry}, M_{rxy}) shown in Figure C-A-N9.3.8 can be checked using interaction equations. These interaction equations were developed based on the conservative simplified design approach developed by Varma et al. (2014), which consists of (i) dividing the SC panel section into two notional halves,

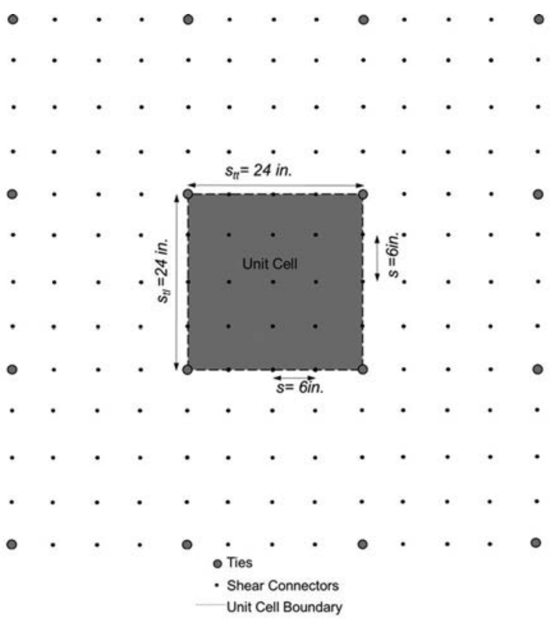


Fig. C-A-N9.3.7. Unit cell for calculating Q_{cv}^{avg}.

(ii) calculating the required in-plane strengths (S'_{rx}, S'_{ry} and S'_{rxy}) for each notional half, and (iii) calculating the required in-plane principal strengths ($S_{r,max}$ and $S_{r,min}$) for each notional half.

Each notional half consists of one faceplate and half the concrete infill thickness as shown in Figure C-A-N9.3.8. The required in-plane strengths (S'_{rx}, S'_{ry} and S'_{rxy}) for each notional half are calculated by representing the out-of-plane moments as force couples with effective arm lengths (for example, 0.90 times the wall thickness for tension dominated situations with significant concrete cracking and 0.67 times the wall thickness for compression dominated situations with limited concrete cracking). The required in-plane principal strengths ($S_{r,max}$ and $S_{r,min}$) can be calculated for each notional half using the required in-plane strengths (S'_{rx}, S'_{ry} and S'_{rxy}) and appropriate equations.

Varma et al. (2014) developed a conservative simplified interaction surface in principal force space for checking the design adequacy of the notional halves of the SC wall panel section. As shown in Figure C-A-N9.3.9, the interaction surface has four regions in principal force space: (i) Region I is for biaxial tension; (ii) Region II is for axial tension plus in-plane shear; (iii) Region III is for axial compression plus in-plane shear; and (iv) Region IV is for biaxial compression.

The interaction surface and these four regions are defined by anchor points located at 50% of the total section strengths in (i) uniaxial tension, (ii) biaxial tension, (iii) pure in-plane shear, (iv) uniaxial compression, and (v) biaxial compression. The 50% reduction reflects that the interaction surface is for each notional half of the SC panel section. The interaction equations for each of these four regions are also provided in Varma et al. (2014).

For further simplification, Regions I and II have been combined into one region described by a straight line connecting the anchor points of pure shear and biaxial tension in the principal force space. This conservatively eliminates the uniaxial tension as an independent anchor point and reduces the number of regions and equations needed for the interaction surface.

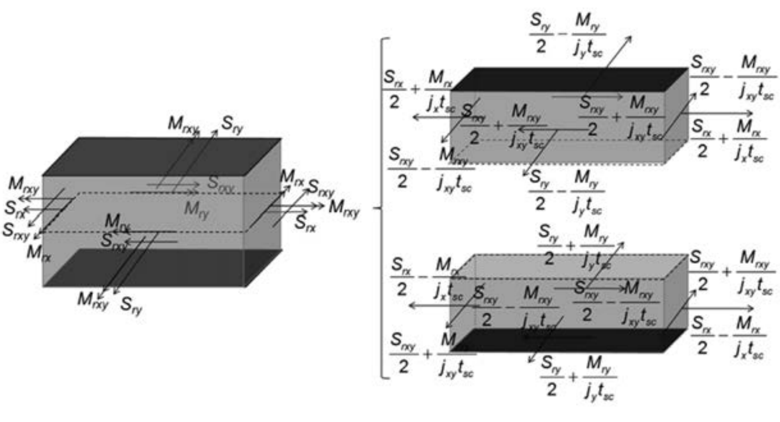


Fig. C-A-N9.3.8. Combined forces acting on panel section and notional halves (Varma et al., 2014).

As shown in Figure C-A-N9.3.10, the uniaxial tensile strength is conservatively adjusted to be collinear with the straight line joining the anchor points of pure in-plane shear and biaxial tension in principal force space. This is always slightly conservative because (i) the pure in-plane shear strength ($V_{ci} = \kappa A_s F_y/2 \le A_s F_y/2$) is always less than or equal to $A_s F_y/2$, (ii) the biaxial tension point is anchored at $A_s F_y/2$, and (iii) $\phi_{vs} = 0.95$ and $\phi_{ti} = 1.00$. Therefore, the resulting unaxial tension anchor point will be slightly less than $A_s F_y/2$.

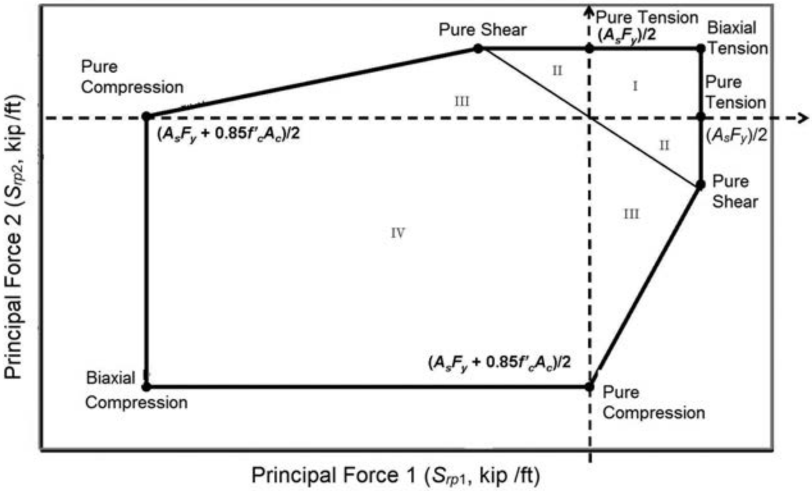


Fig. C-A-N9.3.9. Interaction surface for in-plane forces in principal force space (Varma et al., 2014).

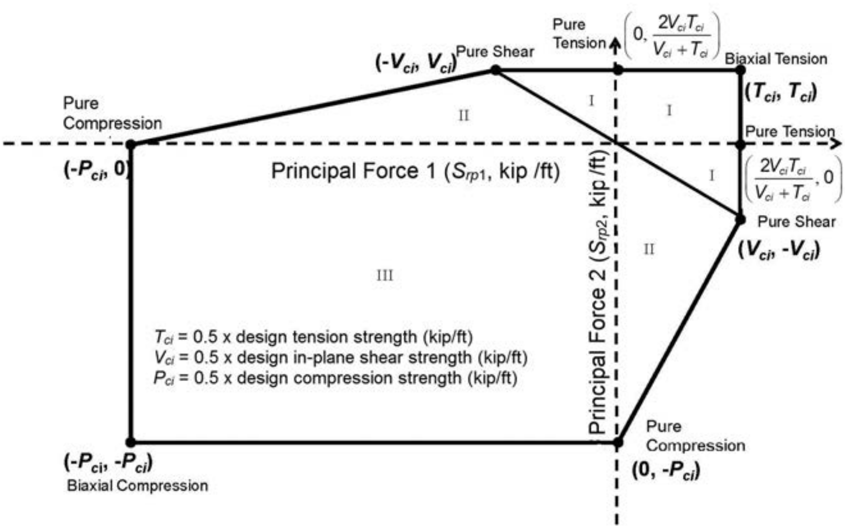


Fig. C-A-N9.3.10. Simplified interaction surface plotted in principal force space.

The resistance and safety factors for available demands for the notional halves have been taken to be less conservative than those for the corresponding individual demands on the panel sections because the maximum individual required tension and shear demands will rarely occur in the same panel section.

Varma et al. (2014) confirmed the conservatism of the design approach by developing a mechanics-based model that accounts for the complex behavior of the composite SC panel section subjected to combined in-plane forces and moments, and also by developing a detailed nonlinear inelastic finite element model of SC panel sections subjected to combined in-plane forces and moments. For example, Figure C-A-N9.3.11 confirms the conservatism of the design approach by comparing the bending moment-in plane shear (M_{rx}, S_{rxy}) interaction predicted for an SC panel section by all three methods: (i) design approach; (ii) mechanics based model; and (iii) finite element model. As shown, the design approach is most conservative.

The alternate interaction Equations A-N9-32 to A-N9-34 were obtained by recasting the interaction Equations A-N9-25 to A-N9-27 (in terms of the principal force $S_{r,max}$ and $S_{r,min}$) directly in terms of S'_{rx}, S'_{ry} and S'_{rxy}. The alternate interaction equations are mathematically equivalent to the interaction equation in terms of the principal forces. This was confirmed algebraically and by plotting points on the interaction surface using both forms of the interaction equations.

For example, Figure C-A-N9.3.12 shows the interaction surface defined by the interaction Equations A-N9-25 to A-N9-27 in terms of the principal forces, and some data points that were obtained using the alternate forms of the interaction Equations A-N9-32 to A-N9-34, which confirms their equivalency. Figure C-A-N9.3.12 was developed using 0.50-in.-thick (13 mm) faceplates made from 50-ksi (350 MPa) yield stress steel filled with 29 in. (725 mm) of 6-ksi (40 MPa) concrete to develop a 30-in.-thick (750 mm) SC wall panel section. The anchor points in Figure C-A-N9.3.12 are without phi factors.

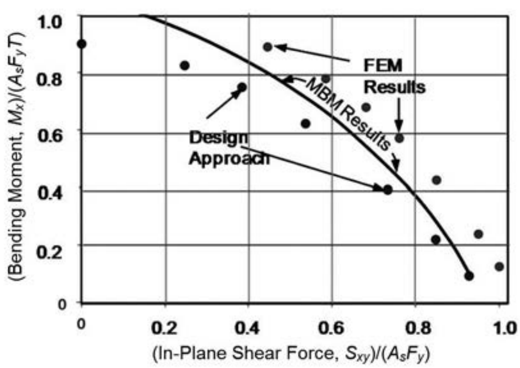


Fig. C-A-N9.3.11. Moment-shear interaction for SC wall (Varma et al., 2014).

N9.4 DESIGN OF SC WALL CONNECTIONS

1. General Provisions

The following connection types are possible: SC wall-to-SC or -RC wall, SC wall-to-RC basemat, SC wall-to-SC or -RC slab. Splices between coplanar SC and RC walls are also possible. Joint constructability and detailing requires careful consideration in SC and composite structures. Bolting and welding are used as connection elements in steel structures; column anchorages involve baseplates, anchor rods and shear lugs. Well established rules and methods exist for sizing these connections. Embedded rebar (dowels), shear-friction rebar, use of joint ties, etc., are used as connection elements across RC-to-RC joints (often construction joints) and again, established rules exist for designing RC connections.

For steel-to-steel connections the following are some general guidelines to follow. Bolts and welds can be sized and installed to provide adequate strength (i.e., match the required strengths or the capacity of the connecting elements). Assuring adequate ductility, especially in seismic applications sometimes requires further consideration and testing to ensure that the connecting elements are able to accommodate large inelastic deformations in the connected members [e.g., post-Northridge research of moment frame connections and ANSI/AISC 358 development (AISC, 2016d)]. For gusseted connections or extended plate connections, simple (empirical) methods exist (e.g., the uniform force method) that are adequate for design instead of having to perform design using complex finite element analyses.

For anchorage of linear steel components, the following are some general guidelines to follow. Linear steel members (e.g., columns) can be anchored into concrete (e.g., basemat) using anchor rods and lugs. This is a case of connection between linear steel members and RC elements (e.g., piers, basemat). Anchor rods are typically used to resist pullout forces and bending moments, while lugs are used to resist shear forces.

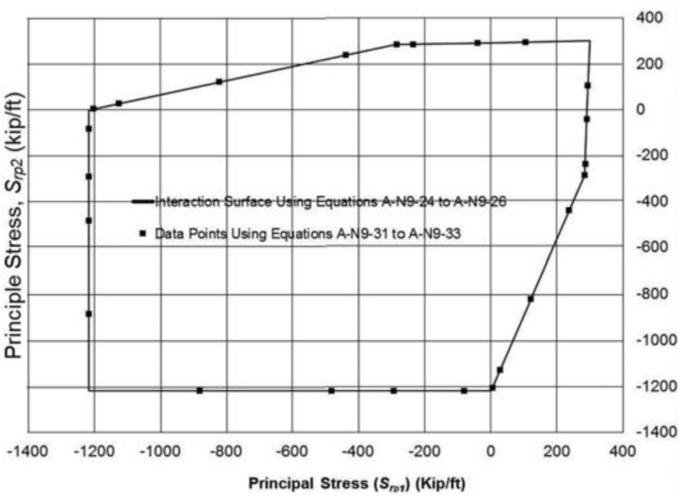


Fig. C-A-N9.3.12. Interaction surface and data points using alternate forms of interaction equations.

Design rules are based on tests that exist for sizing anchor rods [ACI 349-13 or ACI 349M-13, Appendix D (ACI, 2013)] and lugs [AISC Design Guide 2 (Darwin, 1990)]. Demands on connecting elements due to simultaneous forces and moments acting on the anchored member can be determined for their adequate sizing.

For connections to RC elements, the following are some general guidelines to follow:

- Linear or continuum RC elements (e.g., beams/columns and walls/floors) are often connected with other RC elements, usually across construction joints.
- Typical connecting elements are dowels.
- Dowels act as splices for transfer of tension and bending moments; they act as shear-friction reinforcement for transfer of shear forces.
- Closely spaced ties are used to achieve high strain capacity and high shear strength within the beam-column joints.
- A lot of test data and prescriptive design rules exist to adequately size RC connections.

Generally, no prescriptive rules exist for designing connections between linear composite members and RC elements (e.g., filled composite column anchorage). However, various types of connection elements can be used to connect composite members and RC members including the following: pre-tensioned bars or strands, steel-headed stud anchors, dowels, lugs, anchor rods, etc. Possible interaction due to simultaneously acting forces and moments needs to be considered when sizing the connecting elements. The behavior of connecting elements under cyclic loads (e.g., seismic) needs to be considered for ensuring their adequacy.

SC connections are more complicated than connections involving linear composite members as multiple types of demands exist on plate/shell type SC elements. Unlike RC walls, SC walls have very high required in-plane shear strength; use of shear friction reinforcement alone may not be sufficient to match the required strength. Various types of connecting elements may be brought to bear to resist various demands; however, often the same type of connecting element may resist different types of demands simultaneously. Unlike RC member connections, it is not easy to embed the rebar in SC construction because it is in the form of continuous faceplates.

Behavior beyond safe shutdown earthquake performance needs to be considered, especially if the connection involves a brittle failure mode, or if the design needs to satisfy a "Review Level Earthquake." It is possible that the connection will need to be designed to be weaker than the connected elements (particularly for in-plane shear). Adequate inelastic deformation capacity will need to be specified. Interaction due to various types of demands will need to be accounted for, preferably on a small element basis (say two times the SC element thickness) rather than considering the entire SC wall (or SC slab) as one unit.

2. Required Strength

Figure C-A-N9.4.1 lays out the procedure to be followed in calculating the required strength for the connection.

For option (a) (full-strength connections), a load increase factor (*LIF*) of 1.25 has been selected to be consistent with ACI 349-13 or ACI 349M-13 (ACI, 2013) requirements, which is the prevalent code for design of safety-related nuclear concrete facilities. The regulatory agency also considers the precedence established by ACI 349 and ACI 349M to be the relevant rubric for evaluating and accepting SC structures currently being built in the U.S., which are primarily replacements for RC structures. This factor also takes into consideration the strain hardening and overstrength that will be expected in SC walls. Because a full-strength connection is designed for 1.25 times the nominal strength of the connected SC walls, the connection is always adequate, provided the wall is safe for the load combinations considered.

For option (b) (overstrength connections), a *LIF* of 2.0 is applied to the seismic demands with the intention to achieve the minimum high-confidence-of-low-probability-of-failure margin of safety equal to 1.67, while utilizing the approach specified in ACI 349-13 or ACI 349M-13, Appendix D, for the connection design.

3. Available Strength

The connection available strength for each demand type should be calculated using the applicable force transfer mechanism and the available strength of its contributing connectors. Figure C-A-N9.4.2 lays out the procedure to be followed in calculating the available strength for the connection.

Peer review is recommended to determine the connection adequacy for combinations of demands, i.e., combined in-plane and out-of-plane forces. If deemed necessary

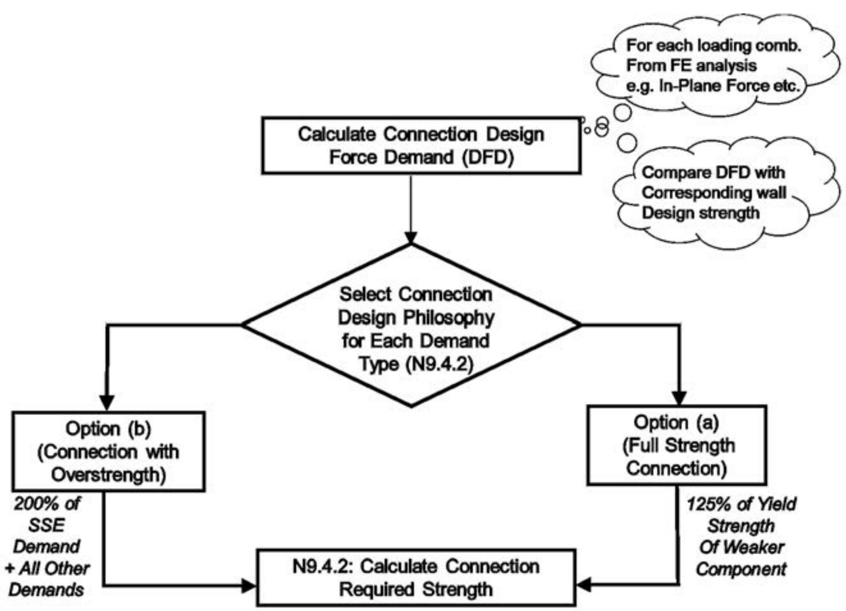


Fig. C-A-N9.4.1. Calculation of connection required strength.

by the peer review, the connection adequacy for combinations of demands should be verified by the results of a nonlinear inelastic finite element analyses conducted using benchmarked nonlinear finite element models. This verification should also be reviewed. Figure C-A-N9.4.3 lays out the procedure for connection qualification.

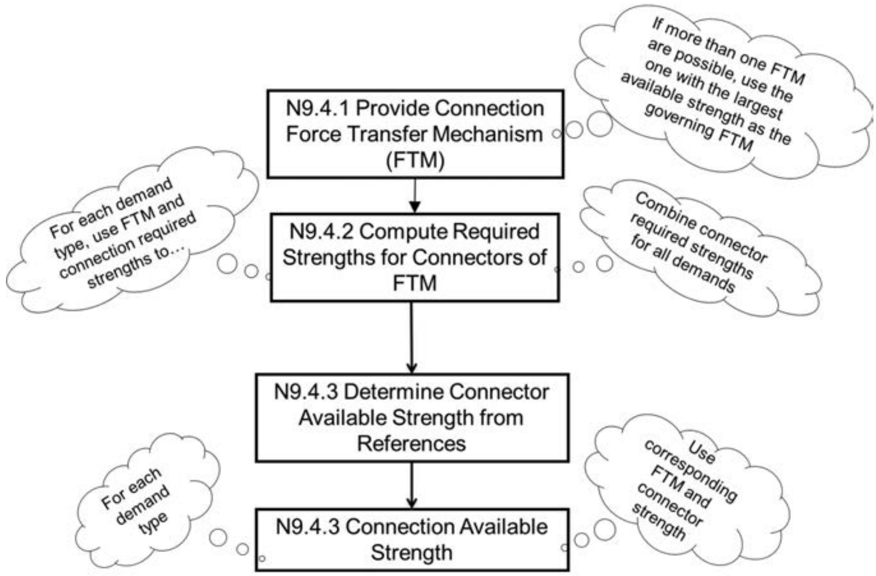


Fig. C-A-N9.4.2. Calculation of connection available strength.

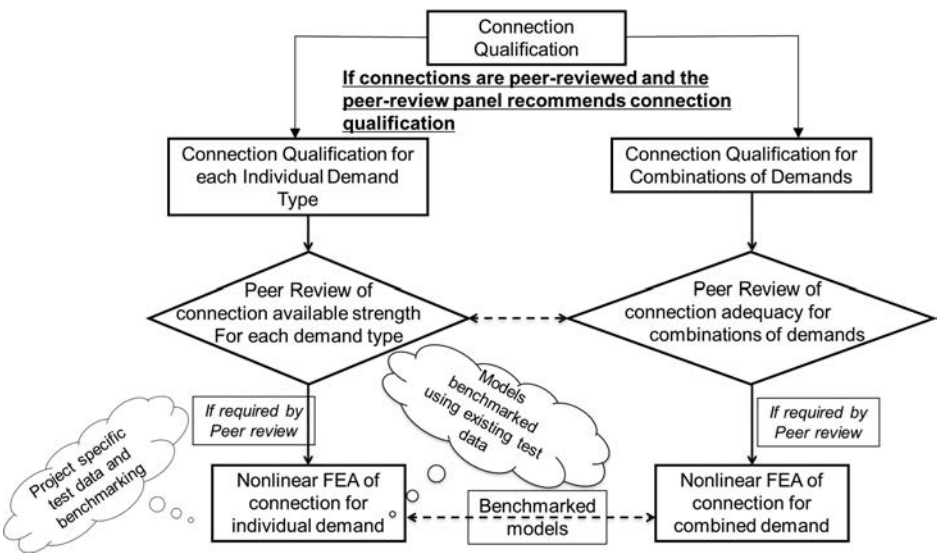


Fig. C-A-N9.4.3. Connection qualification.

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