ANSI/AISC N690-24
An American National Standard

Specification for Safety-Related Steel Structures for Nuclear Facilities

October 4, 2024

Supersedes the *Specification for Safety-Related Steel Structures for Nuclear Facilities*, dated June 28, 2018, 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 N690-24 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 2022 *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*.

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

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

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

Symbol	Definition	Section
F_{nr}	Nominal rupture strength of the tie, or the nominal strength of the associated welded or threaded connection, whichever is smaller, kips (N)	App. N9.1.5a
F_{ny}	Nominal yield strength of the tie based on its gross area if no threads are present, and on its root area if it is threaded, kips (N)	App. N9.1.5a
F_t	Nominal tensile strength of tie, kips (N)	App. N9.3.5
F_u	Specified minimum tensile strength, ksi (MPa)	NJ3.11
F_y	Specified minimum yield stress, ksi (MPa). As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	App. N9.1.1
$(GA)_{eff}$	Effective in-plane shear stiffness per unit width, kip/ft (N/m)	App. N9.2.2
$(GA)_{uncr}$	In-plane shear stiffness per unit width of uncracked composite SC panel section, kip/ft (N/m)	App. N9.2.2
G_c	Shear modulus of concrete, $= 772\sqrt{f'_c}$, ksi $\left(2\,000\sqrt{f'_c}, \text{MPa}\right)$	App. N9.2.2
G_s	Shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa) for carbon steel and duplex stainless steel = 10,800 ksi (74 500 MPa) for austenitic stainless steel	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 per unit width of faceplates (corresponding to the condition when the concrete is fully cracked), in.^{4}/ft (mm^4/m)	App. N9.2.2
K_p	Strength-dependent concrete penetrability factor	App. N10.3.2
K_{psc}	Penetration depth modification factor for SC cross section	App. N10.3.2
L	Live load due to occupancy and moveable equipment, including impact	NB2.1
L_c	Effective length of member, in. (mm)	App. N9.3.2
L_d	Development length, in. (mm)	App. N9.1.4b
L_r	Roof live load	NB2.1
M_n	Nominal flexural strength per unit width of SC structural element, kip-in./ft (N-mm/m)	App. N9.1.4b
M_{rx}, M_{ry}	Required out-of-plane flexural strength per unit width, kip-in./ft (N-mm/m)	App. N9.2.4b
M_{rxy}	Required twisting moment strength per unit width, kip-in./ft (N-mm/m)	App. N9.2.5
N	Missile nose shape factor per the modified NDRC formula	App. N10.3.2
P_a	Maximum differential pressure load generated by postulated accident	NB2.4
P_{ci}	Available compressive strength per unit width for each notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
P_e	Elastic critical buckling load per unit width, kip/ft (N/m)	App. N9.3.2
P_{no}	Nominal compressive strength per unit width, kip/ft (N/m)	App. N9.3.2

Symbol	Definition	Section
Q_{cv}	Available shear strength, kips (N)	App. N9.1.4a
Q_{cv}^{avg}	Weighted average of the available interfacial shear strengths of a group of shear connectors, kips (N)	App. N9.1.4b
Q_{cv}^{avg}	Weighted average of the available interfacial shear strengths of a group of shear connectors that accounts for tributary areas of each type of connector, kips (N)	App. N9.3.6a
R	Rain load	NB2.1
R_a	Pipe and equipment reactions generated by a postulated accident, including R_o	NB2.4
R_a	Required strength using ASD load combinations	NB2.6
R_n	Nominal strength	NB2.5
R_o	Pipe reactions during normal operating, start-up, or shutdown conditions, based on most critical transient or steady-state condition	NB2.1
R_u	Required strength using LRFD load combinations	NB2.5
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y, of that material	App. N10.2.1
S	Snow load	NB2.1
S_{cr}	In-plane shear force per unit width at concrete cracking threshold, kip/ft (N/m)	App. N9.2.2
$S_{r,max}$	Maximum required principal in-plane strength per unit width for notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
$S_{r,min}$	Minimum required principal in-plane strength per unit width for notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
S_{rx}	Required membrane axial strength per unit width in direction x, kip/ft (N/m)	App. N9.2.5
S_{rxy}	Required membrane in-plane shear strength per unit width, kip/ft (N/m)	App. N9.2.2
S_{ry}	Required membrane axial strength per unit width in direction y, kip/ft (N/m)	App. N9.2.5
S'_{rx}	Required membrane axial strength per unit width in direction x for each notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
S'_{rxy}	Required membrane in-plane shear strength per unit width for each notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
S'_{ry}	Required membrane axial strength per unit width in direction y for each notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
T_a	Thermal loads generated by a postulated accident, including T_o	NB2.4
T_{ci}	Available tensile strength per unit width for each notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
T_{ni}	Nominal tensile strength per unit width of SC panel section, kip/ft (N/m)	App. N9.3.6b
T_o	Thermal effects and loads during normal operating, start-up, or shutdown conditions, based on most critical transient or steady-state condition	NB2.1
V_c	Available out-of-plane shear strengths per unit width of SC panel section in local x (V_{cx}) and y (V_{cy}) directions, kip/ft (N/m)	App. N9.3.6a

Symbol	Definition	Section
$V_{c\ conc}$	Available out-of-plane shear strength contributed by concrete per unit width of SC panel section, kip/ft (N/m)	App. N9.3.6a
V_{ci}	Available in-plane shear strength per unit width for each notional half of SC panel section, kip/ft (N/m)	App. N9.3.6b
V_{conc}	Nominal out-of-plane shear strength contributed by concrete per unit width of SC panel section, kip/ft (N/m)	App. N9.3.5
V_i	Initial (pre-impact) velocity of missile, ft/s (m/s)	App. N10.3.2
V_{ni}	Nominal in-plane shear strength per unit width of SC panel section, kip/ft (N/m)	App. N9.3.4
V_{no}	Nominal out-of-plane shear strength per unit width of SC panel section, kip/ft (N/m)	App. N9.3.5
V_p	Perforation velocity for reinforced concrete section of same thickness, ft/s (m/s)	App. N10.3.2
V_r	Required out-of-plane shear strength per unit width of SC panel section in local x (V_{rx}) and y (V_{ry}) directions using LRFD or ASD load combinations, kip/ft (N/m)	App. N9.3.6a
V_r	Residual velocity of a missile passing through concrete, ft/s (m/s)	App. N10.3.2
V_{rx}	Required out-of-plane shear strength per unit width along edge parallel to direction x, kip/ft (N/m)	App. N9.2.5
V_{ry}	Required out-of-plane shear strength per unit width along edge parallel to direction y, kip/ft (N/m)	App. N9.2.5
V_s	Nominal out-of-plane shear strength contributed by steel per unit width of SC panel section, kip/ft (N/m)	App. N9.3.5
W	Wind load	NB2.2
W_{cf}	Weight of the concrete frustum (plug) associated with x_{sc}, the penetration depth, of the impacting missile, lb (N)	App. N10.3.2
W_p	Missile weight, lb (N)	App. N10.3.2
W_t	Loads generated by the specified design (basis) tornado, including wind pressures, pressure differentials, and tornado-borne missiles	NB2.3
Y_j	Jet impingement load generated by the postulated accident	NB2.4
Y_m	Missile impact load, such as pipe whip generated by or during the postulated accident	NB2.4
Y_r	Loads on structure generated by reaction of a broken high-energy pipe during a postulated accident	NB2.4
b	Largest unsupported length of the faceplate between rows of shear connectors, in. (mm)	App. N9.1.3
b	Width of compression element as shown in Table A-N10.2.1, in. (mm)	Table A-N10.2.1
c_2	Calibration constant for determining effective flexural stiffness	App. N9.2.2
c_m	As-modeled specific heat used in elastic finite element analysis of SC panel section, Btu/lb-°F (J/kg-°C)	App. N9.2.3
d	Nominal diameter of fastener, in. (mm)	NJ3.11
d	Full depth of the section, for stems of tees, in. (mm)	Table A-N10.2.1

Symbol	Definition	Section
d	Effective diameter of the missile, in. (mm)	App. N10.3.2
f_w	Faceplate waviness, in. (mm)	NM2.7
f'_c	Specified compressive strength of concrete, ksi (MPa)	Table NA-4.2.2
$f'_c(T)$	Specified compressive strength of concrete at elevated temperature, ksi (MPa)	Table NA-4.2.2
g	Acceleration due to gravity = 386.4 in./s^2 (9.81m/s^2)	App. N10.3.2
h	Width of compression element as shown in Table A-N10.2.1, in. (mm)	Table A-N10.2.1
j_x, j_y	Parameter for distributing required flexural strength into the corresponding membrane force couples acting on each notional half of SC panel section	App. N9.3.6b
j_{xy}	Parameter for distributing required flexural strength, M_{rxy}, into the corresponding membrane force couples acting on each notional half of SC panel section	App. N9.3.6b
k_{Ec}, k_c	Retention factors for concrete	Table NA-4.2.2
k_m	As-modeled thermal conductivity used in the elastic finite element analysis of SC panel sections, Btu/ft-sec-°F (W/m-°C)	App. N9.2.3
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)	NJ3.11
n	Modular ratio of steel and concrete	App. N9.2.2
r	Radius of gyration, in. (mm)	Table A-N10.2.2
r_1	Effective radius of the missile, in. (mm)	App. N10.3.2
r_2	Concrete frustum radius at the inside face of the back faceplate, in. (mm)	App. N10.3.2
s	Spacing of steel anchors, in. (mm)	NM2.7
s	Spacing of shear connectors, in. (mm)	App. N9.3.6a
s_{tl}	Spacing of ties in the longitudinal direction, in. (mm)	App. N9.1.5b
$s_{t,min}$	Minimum tie spacing, in. (mm)	NM2.7
s_{tt}	Spacing of ties in the transverse direction, in. (mm)	App. N9.1.5b
t	Thickness of connected material, in. (mm)	NJ3.11
t	Thickness of element as shown in Table A-N10.2.1, in. (mm)	Table A-N10.2.1
t_c	Concrete infill thickness, in. (mm)	App. N9.2.2
t_m	As-modeled section thickness of SC panel section, in. (mm)	App. N9.2.3
t_p	Thickness of faceplate, in. (mm)	NM2.7
$t_{p,min}$	Minimum required faceplate thickness, in. (mm)	App. N10.3.2
t_{sc}	SC section thickness, in. (mm)	App. N9.1.1
t_w	Thickness of web, in. (mm)	Table A-N10.2.1
x_c	Concrete penetration depth for the reinforced concrete section of the same thickness as the SC cross section, in. (mm)	App. N10.3.2
x_{sc}	Missile penetration depth into the SC cross section, in. (mm)	App. N10.3.2
w_c	Weight of concrete per unit volume ($90 \le w_c \le 155$ lb/ft^3 or $1\,500 \le w_c \le 2\,500$ kg/m^3)	App. N9.2.2

Symbol	Definition	Section
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, °F (°C)	App. N9.2.4b
Ω	Safety factor	NB2.6
Ω_{ci}	Safety factor for compression for each notional half	App. N9.3.6b
Ω_{ti}	Safety factor for tension for each notional half	App. N9.3.6b
Ω_{vi}	Safety factor for in-plane shear	App. N9.3.4
Ω_{vo}	Safety factor for out-of-plane shear	App. N9.3.5
Ω_{vs}	Safety factor for in-plane shear for each notional half	App. N9.3.6b
α_m	As-modeled thermal expansion coefficient used in the elastic finite element analysis of SC panel section, °F^{-1} (°C^{-1})	App. N9.2.3
α_p	Missile deformability factor	App. N10.3.2
α_s	Thermal expansion coefficient of faceplate, °F^{-1} (°C^{-1})	App. N9.2.4b
γ_m	As-modeled material density used in elastic finite element analysis of the SC panel section, lb/ft^3 (kg/m^3)	App. N9.2.3
$\varepsilon_{cu}(T)$	Concrete strain corresponding to $f'_c(T)$ at elevated temperature	Table NA-4.2.2
ε_{st}	Strain corresponding to the onset of strain hardening	Table A-N10.2.2
ε_u	Strain corresponding to elongation at failure (rupture)	Table A-N10.2.2
ε_y	Strain corresponding to nominal yield stress	Table A-N10.2.2
θ	Inclination angle of the concrete frustum extending from the penetration depth of the impacting missile to the back faceplate of the impacted SC section, degrees	App. N10.3.2
λ_c	Limiting width-to-thickness ratio for compression elements due to impactive and impulsive load	App. N10.2.1
μ_p	Permissible ductility ratio	App. N10.2.4
μ_r	Required ductility ratio	App. N10.2.4
ν_m	As-modeled Poisson's ratio used in elastic finite element analysis of the SC panel section	App. N9.2.3
ξ	Factor used to calculate shear reinforcement contribution to out-of-plane shear strength	App. N9.3.5
ρ	Reinforcement ratio	App. N9.1.1
ρ_c	Concrete density, lb/ft^3 (kg/m^3)	App. N10.3.2
ρ'	Stiffness-adjusted reinforcement ratio	App. N9.2.2
$\overline{\rho}$	Strength-adjusted reinforcement ratio	App. N9.2.2
σ_r	Equivalent radial compressive stress in the rear faceplate, based on von Mises yield criterion, ksi (MPa)	App. N10.3.2
ϕ	Resistance factor	NB2.5
ϕ_{ci}	Resistance factor for compression for each notional half	App. N9.3.6b
ϕ_{ti}	Resistance factor for tension for each notional half	App. N9.3.6b
ϕ_{vi}	Resistance factor for in-plane shear	App. N9.3.4
ϕ_{vo}	Resistance factor for out-of-plane shear	App. N9.3.5
ϕ_{vs}	Resistance factor for in-plane shear for each notional half	App. N9.3.6b

GLOSSARY

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

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

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

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

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

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

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

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

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

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

Ductile limit state. Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width-to-thickness limitations of Table A-N10.2.1. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.

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

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

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

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

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

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

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

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

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

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

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

Jet impingement load. Force-time history depicting the forces resulting from the direct strike by a dense, high-velocity jet of steam or water onto a structure, system, or component.

Jet shield. Device used to protect adjacent structures, systems, or components from the effects of a dense, high-velocity jet of steam or water, resulting from the rupture of a high-energy pipeline.

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

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

Module. A combination of sub-modules.

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

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

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

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

Operating basis earthquake (OBE). Earthquake that produces vibratory ground motion for which those features of the nuclear power plant necessary for continued operation

without undue risk to the health and safety of the public will remain functional. Unless elected by the owner as a design input, the OBE is only associated with plant shutdown and inspection.

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

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

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

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

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

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

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

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

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

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

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

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

(1) The integrity of the reactor coolant pressure boundary;
(2) The capability to shut down the reactor and maintain it in a safe shut down condition; or
(3) The capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10CFR100.

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

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

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

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

Specified design (basis) tornado. Combination of translational speed, rotational speed, and prescribed pressure drop related to the environmental effects of a tornado [as defined by the licensing basis, design basis, and/or regulatory requirements; for example, U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.76].

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

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

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

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

ABBREVIATIONS

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

ABC (applicable building code)

ACI (American Concrete Institute)

AHJ (authority having jurisdiction)

AISC (American Institute of Steel Construction)

ANSI (American National Standards Institute)

ASCE (American Society of Civil Engineers)

ASD (allowable strength design)

ASME (American Society of Mechanical Engineers)

ASNT (American Society for Nondestructive Testing)

ASTM (ASTM International)

AWI (associate welding inspector)

AWS (American Welding Society)

CFR (U.S. Code of Federal Regulations)

CJP (complete joint penetration)

CMAA (Crane Manufacturers Association of America)

CMTR (certified material test report)

CVN (Charpy V-notch)

DBE (design basis earthquake or design/evaluation basis earthquake)

DIF (dynamic increase factor)

DLF (dynamic load factor)

DOE (U.S. Department of Energy)

EOR (engineer of record)

HSS (hollow structural section)

HVAC (heating, ventilation and air conditioning)

LOCA (loss-of-coolant accident)

LRFD (load and resistance factor design)

MT (magnetic particle testing)

NDE (nondestructive examination)

NRC (U.S. Nuclear Regulatory Commission)

OBE (operating basis earthquake)

PJP (partial joint penetration)

PQR (procedure qualification record)

PT (penetrant testing)

QA (quality assurance)

QAI (quality assurance inspector)

QC (quality control)

QCI (quality control inspector)

RC (reinforced concrete)

RCSC (Research Council on Structural Connections)

RT (radiographic testing)

SC (steel-plate composite)

SEI (Structural Engineering Institute)

SSE (safe shutdown earthquake)

SWI (senior welding inspector)

UT (ultrasonic testing)

WI (welding inspector)

WPQR (welding personnel performance qualification records)

WPS (welding procedure specification)

CHAPTER NA

GENERAL PROVISIONS

Modify Chapter A of the Specification as follows.

Replace preamble with the following:

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

The chapter is organized as follows:

NA1. Scope
NA2. Referenced Specifications, Codes, and Standards
NA3. Material
NA4. Structural Design Documents and Specifications
NA5. Approvals
NA6. Quality Assurance

NA1. SCOPE

Replace section with the following:

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

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

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

The Nuclear Specification includes the list of Symbols, Glossary terms, Abbreviations, Chapters NA through NN, and Appendices N1 through N10. The Commentary and User Notes interspersed throughout the Nuclear Specification are not part of the Nuclear Specification. The phrases "is permitted" and "are permitted" in this document identify provisions that comply with the Nuclear Specification, but are not mandatory.

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

User Note: With the exception of Appendix N9, this standard does not include seismic detailing requirements for safety-related nuclear structures constructed using structural steel and composite members. The authority having jurisdiction may adopt the pertinent requirements in ASCE 43.

For steel-plate composite (SC) structural elements and their connections, the design and detailing requirements specified in Appendix N9 are adequate for seismic applications.

The steel elements shall be as defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303), Section 2.1, hereafter referred to as the *Code of Standard Practice*.

Structures and structural elements subject to the Nuclear Specification are those steel structures and structural elements that are part of a safety-related system or that support, house, or protect safety-related systems or components, the failure of which could credibly result in the loss of capability of the structure, system, or component to perform its safety functions. Concrete that is part of composite members and steel-plate composite (SC) structural elements is also subject to the Nuclear Specification. Safety categorization for nuclear facility steel structures and structural elements shall be the responsibility of the owner and shall be identified in the contract documents.

Specifically excluded from the Nuclear Specification are the pressure-retaining components, including, but not limited to, pressure vessels, valves, pumps, and piping.

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

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

User Note: With the exception of hollow structural sections (HSS), for the design of structural members that are cold-formed to shapes with elements not more than 1 in. (25 mm) in thickness, the use of provisions of the ANSI/AISI S100 *North American Specification for the Design of Cold-Formed Steel Structural Members* is recommended, incorporating the loads and load combinations delineated in Section NB2.

NA2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

Add the following:

Crane Manufacturers Association of America (CMAA)

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

U.S. Nuclear Regulatory Commission

NUREG-0800 *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants*, March 2007

U.S. Code of Federal Regulations (CFR)

Title 10 of the *Code of Federal Regulations*, Part 50 (10CFR50), Appendix B 2019, and Appendix S, 2020

Title 10 of the *Code of Federal Regulations*, Part 830, Subpart A, Quality Assurance Requirements (to be used for Department of Energy Nuclear Facilities), 2020

Title 10 of the *Code of Federal Regulations*, Part 100 (10CFR100), Reactor Site Criteria, 2019

U.S. Department of Energy (DOE)

DOE Order O 414.1D, *Quality Assurance*, April 2011

Nuclear Energy Institute (NEI)

NEI 07-13, *Methodology for Performing Aircraft Impact Assessments for New Plant Designs*, Revision 8P, 2011

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

ACI 117-10 *Specification for Tolerances for Concrete Construction and Materials and Commentary*

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

A1008/A1008M-21a *Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability, Required Hardness, Solution Hardened, and Bake Hardenable*

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

AWS A5.4/A5.4M:2012(R2022) *Specification for Stainless Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.9/A5.9M:2022 *Welding Consumables-Wire Electrodes, Strip Electrodes, Wires, and Rods for Arc Welding of Stainless and Heat Resisting Steels—Classification*

AWS A5.22/A5.22M:2012 *Specification for Stainless Steel Flux Cored and Metal Cored Welding Electrodes and Rods*

AWS D1.4/D1.4M:2018-AMD1 *Structural Welding Code—Steel Reinforcing Bars*

AWS D1.6/D1.6M:2017-AMD1 *Structural Welding Code—Stainless Steel*

AWS D1.8/D1.8M:2021 *Structural Welding Code—Seismic Supplement*

NA3. MATERIAL

1. Structural Steel Materials

Replace section with the following:

In addition to satisfying the applicable ASTM standards, the specification of the material of those structures or structural components that are subject to impactive and/or impulsive loads shall be supplemented by the requirement that the material be subjected to Charpy V-notch (CVN) impact tests, using the procedures described in ASTM A673/A673M. The CVN impact test shall be conducted at a temperature of 0°F (–18°C). For plates and structural shapes with plate thicknesses and flange

TABLE NA3.1
Charpy V-Notch Energy Values

Specified Minimum Yield Stress	Charpy V-Notch Energy Value	
	Average of Three Specimens, Minimum	One Individual Specimen, Minimum
36 ksi (250 MPa) up to and including 65 ksi (450 MPa)	25 ft-lb (34 J)	20 ft-lb (27 J)
Matching 70 ksi (485 MPa) and 80 ksi (550 MPa) weld filler metal	25 ft-lb (34 J)	20 ft-lb (27 J)

thicknesses, respectively, equal to or less than 2 in. (50 mm) and for weld metal, the acceptance criteria shall be based on energy values indicated in Table NA3.1, in addition to satisfying the applicable ASTM and AWS standard.

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

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

Certified material test reports (CMTR) or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material conforms with one of the ASTM standards listed in *Specification* Table A3.1, subject to the grades and limitations listed, and meets the CVN requirements of Table NA3.1.

1a. Listed Materials

Modify Table A3.1 as follows.

(b) Hollow structural sections (HSS)

Add the following:

ASTM A106/A106M
ASTM A312/A312M

(c) Plates

Add the following:

ASTM A240/A240M
ASTM A515/A515M
ASTM A516/A516M

ASTM A537/A537M Class 1 and Class 2
ASTM A709/A709M
ASTM A738/A738M Grades B and C

(d) Bars

Add the following:

ASTM A276
ASTM A479/A479M

(e) Sheet

Add the following:

ASTM A666
ASTM A1008/A1008M

Add the following:

For the design of structural members cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steels, refer to Chapters 3 through 10 of ASCE/SEI 8. ASCE/SEI 8 is not applicable for hot-rolled or built-up steel members, assemblies, and connections.

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

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

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

1c. Unidentified Steel

Replace section with the following:

Unidentified steel shall not be used.

1d. Rolled Heavy Shapes

Add the following:

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

User Note: 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).

1e. Built-Up Heavy Shapes

Add the following:

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

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

(a) Reduce the volume of weld metal to the extent practical.

(b) Select materials that are resistant to lamellar tearing.

(c) Perform through thickness tension testing in accordance with ASTM A770/A770M-03(2018), *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-17, *Standard Specification for Ultrasonic Angle-Beam Examination of Steel Plates*, or A578/A578M-17, *Standard Specification for Straight-Beam Ultrasonic Examination of Rolled 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 ultrasonic testing (UT) examination after the inlay or overlay but prior to making the welded joint.

3. Bolts, Washers, and Nuts

(a) Bolts

Add the following:

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

5. Consumables for Welding

Replace section with the following:

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

AWS A5.1/A5.1M
AWS A5.4/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 A5.32M/A5.32

CVN requirements are provided in Section NJ2.6.

6. Headed Stud Anchors

Replace section with the following:

Steel headed stud anchors shall conform to the requirements of *Structural Welding Code—Steel*, AWS D1.1/D1.1M.

User Note: Studs are typically made from cold drawn bar conforming to the requirements of ASTM A108-18, *Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished*, standard quality, Grades 1010 through 1020, inclusive, either semi-killed or killed aluminum or silicon deoxidation.

Add the following section:

7. Material Certification

Certified material test reports (CMTR) or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material meets the applicable specification.

NA4. STRUCTURAL DESIGN DOCUMENTS AND SPECIFICATIONS

1. Structural Design Documents and Specifications Issued for Construction

Add the following:

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

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

The structural design documents and specifications shall include:

(1) Applicable code references

(2) Material specifications

(3) Material shipping, handling, and storage requirements

(4) Surface preparation and protective coating requirements

(5) Requirements for fabrication and/or erection

(6) Welding and bolting requirements

(7) Tests and inspection requirements

(8) Requirements for shop drawings

(9) Documentation and retention of records

(10) Identify complete-joint-penetration groove welds to be 100% inspected by either ultrasonic testing (UT) or radiographic testing (RT).

Add the following section:

NA6. QUALITY ASSURANCE

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

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

User Note: 10CFR50, Appendix B, and 10CFR830, Subpart A, provide regulations for quality assurance (QA) and quality control (QC). Both these documents defer many requirements to ASME NQA-1. The requirements of Chapter NN are aimed to further assist the user in developing a QA/QC program that will satisfy these regulations for safety-related structural steel, composite, and steel-plate composite (SC) structures.

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

CHAPTER NB

DESIGN REQUIREMENTS

Modify Chapter B of the Specification as follows.

Replace preamble with the following:

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

The chapter is organized as follows:

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
NB8. Dimensional Tolerances

NB2. LOADS AND LOAD COMBINATIONS

Replace section with the following:

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

1. Normal Loads

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

D = dead loads due to the weight of the structural elements, fixed-position equipment, and other permanent appurtenant items; weight of crane trolley and bridge
C = rated capacity of crane (shall include the maximum wheel loads of the crane and the vertical, lateral, and longitudinal forces induced by the moving crane)
F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights
H = loads due to weight and pressure of soil, water in soil, or bulk materials
L = live load due to occupancy and moveable equipment, including impact
L_r = roof live load
R = rain load
R_o = pipe reactions during normal operating, start-up, or shutdown conditions, based on the most critical transient or steady-state condition
S = snow load
T_o = thermal effects and loads during normal operating, start-up, or shutdown conditions, based on the most critical transient or steady-state condition

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

2. Severe Environmental Loads

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

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

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

User Note: The OBE is an earthquake that could reasonably be expected to occur at the plant site during the operating life of the plant considering the regional and local geology, and seismology and specific characteristics of local subsurface material. It is that earthquake that produces the vibratory ground motion for which the features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional.

3. Extreme Environmental Loads

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

E_s = loads generated by the safe shutdown earthquake (SSE) or design basis earthquake (DBE)
W_t = loads generated by the specified design (basis) tornado, including wind pressures, pressure differentials, and tornado-borne missiles

Safe shutdown earthquake loads shall be as defined in 10CFR50, Appendix S, or as specified by the AHJ. Tornado-based loads shall be as defined in the U.S. Nuclear Regulatory Commission Standard Review Plan Section 3.3.2 (NUREG-0800) or as specified by the AHJ.

4. Abnormal Loads

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

P_a = maximum differential pressure load generated by the postulated accident
R_a = pipe and equipment reactions generated by the postulated accident, including R_o
T_a = thermal loads generated by the postulated accident, including T_o
Y_j = jet impingement load generated by the postulated accident
Y_m = missile impact load, such as pipe whip generated by or during the postulated accident
Y_r = loads on the structure generated by the reaction of the broken high-energy pipe during the postulated accident

5. Load and Resistance Factor Design (LRFD)

The design strength, ϕR_n, of each structural component, where R_n is the nominal strength and ϕ is the resistance factor, shall be equal to or greater than the required strength, R_u, determined from the applicable critical combinations of the loads. The possibility of one or more loads not acting concurrently shall be considered when determining the load combination(s) that produce the most critical structural effects. The load combinations specified in this section shall be investigated.

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

5a. Normal Load Combinations

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

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

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

5b. Severe Environmental Load Combinations

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

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

5c. Extreme Environmental and Abnormal Load Combinations

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

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

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

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

5d. Other Considerations

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

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

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

the Commentary for additional guidance regarding analysis of load effects due to T_a including benefits of using the direct analysis method described in Chapter C of the *Specification*.

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

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

(4) If the dead load acts to stabilize the structure against the destabilizing effects of lateral force or uplift, the load factor on dead load shall be 0.90 of the assigned factor, and that on other gravity loads (L, L_r, S, C) shall be zero provided the load does not contribute to the destabilizing effect. F shall be treated in the same manner as D, and H shall be treated in the same manner as L when stability evaluations are performed.

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

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

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

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

6. Allowable Strength Design (ASD)

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

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

6a. Normal Load Combinations

$$D + L + R_o + F + H + T_o + C \tag{NB2-10}$$

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

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

6b. Severe Environmental Load Combinations

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

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

6c. Extreme Environmental and Abnormal Load Combinations

$$D + L + C + R_o + T_o + E_s + F + H \tag{NB2-15}$$

$$D + L + R_o + T_o + W_t + F + H \tag{NB2-16}$$

$$D + L + C + P_a + R_a + T_a + F + H \tag{NB2-17}$$

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

6d. Other Considerations

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

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

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

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

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

(4) If the dead load acts to stabilize the structure against the destabilizing effects of lateral force or uplift, the load factor on dead load shall be 0.60 and other gravity loads (L, L_r, S, C) shall be assumed to equal zero provided the load does not contribute to the destabilizing effect. F shall be treated in the same manner as D, and H shall be treated in the same manner as L when stability evaluations are performed.

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

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

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

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

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

NB3. DESIGN BASIS

***Add the following*:**

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

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

Add the following:

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

3. Required Strength

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

The required strength of structural members and connections shall be determined by structural analysis for the applicable load combinations stipulated in Section NB2.

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

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

8. Design for Serviceability

Add the following:

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

Add the following section:

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

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

NB5. FABRICATION AND ERECTION

Replace section with the following:

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

NB6. QUALITY CONTROL AND QUALITY ASSURANCE

Replace section with the following:

Quality control and quality assurance activities shall satisfy the requirements stipulated in Section NA6, Quality Assurance, and Chapter NN, Quality Control and Quality Assurance.

NB7. EVALUATION OF EXISTING STRUCTURES

Replace section with the following:

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

CHAPTER NC

DESIGN FOR STABILITY

Modify Chapter C of the Specification as follows.

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

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

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

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

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

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

CHAPTER ND

DESIGN OF MEMBERS FOR TENSION

No changes to Chapter D of the Specification.

CHAPTER NE

DESIGN OF MEMBERS FOR COMPRESSION

No changes to Chapter E of the Specification.

CHAPTER NF

DESIGN OF MEMBERS FOR FLEXURE

No changes to Chapter F of the Specification.

CHAPTER NG

DESIGN OF MEMBERS FOR SHEAR

No changes to Chapter G of the Specification.

CHAPTER NH

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

No changes to Chapter H of the Specification.

CHAPTER NI

DESIGN OF COMPOSITE MEMBERS

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

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

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

Modify Chapter I of the Specification as follows.

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

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

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

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

CHAPTER NJ

DESIGN OF CONNECTIONS

Modify Chapter J of the Specification as follows.

NJ1. GENERAL PROVISIONS

Modify section as follows.

Replace Section J1.9 with the following:

9. Welded Alterations to Structures with Existing Rivets or Bolts

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

Replace Section J1.10 with the following:

10. High-Strength Bolts in Combination with Existing Rivets

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

NJ2. WELDS AND WELDED JOINTS

Modify section as follows.

6. Filler Metal Requirements

Replace second paragraph with the following:

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

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

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

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

NJ3. BOLTS, THREADED PARTS, AND BOLTED CONNECTIONS

Modify section as follows.

2. High-Strength Bolts

Add the following to paragraph (b):

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

(5) Other connections stipulated on the design documents

Add the following to paragraph (c):

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

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

11. Bearing and Tearout Strength at Bolt Holes

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

(a) Bearing

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

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

(b) Tearout

$$R_n = 1.2l_ctF_u \tag{J3-6c}$$

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

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

$$R_n = 1.2l_ctF_u \tag{J3-6g}$$

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

Add the following new section:

14. Connections for Members Subject to Impactive or Impulsive Loads

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

CHAPTER NK

ADDITIONAL REQUIREMENTS FOR HSS AND BOX-SECTION CONNECTIONS

No changes to Chapter K of the Specification.

CHAPTER NL

DESIGN FOR SERVICEABILITY

Modify Chapter L of the Specification as follows.

Replace preamble with the following:

This chapter addresses serviceability design requirements.

The chapter is organized as follows:

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

NL1. GENERAL PROVISIONS

Replace section with the following:

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

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

CHAPTER NM

FABRICATION AND ERECTION

Modify Chapter M of the Specification as follows.

NM1. FABRICATION AND ERECTION DOCUMENTS

Replace section with the following:

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

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

Fabrication and erection documents shall have a means of indicating which parts are safety-related.

NM2. FABRICATION

1. Cambering, Curving, and Straightening

Modify section to read as follows:

Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature, and straightness. The temperature of heated areas shall not exceed the lesser of the maximum specified in the applicable ASTM standard or 1,200°F (650°C) for carbon steels. For ASTM A514/A514M and ASTM A709/A709M Grade 70, the temperature of heated areas shall not exceed 1,100°F (590°C). The temperature of heated areas for ferritic, martensitic, or duplex stainless steels shall not exceed 600ºF (320°C). The temperature of heated areas for austenitic stainless steel shall not exceed 800ºF (430°C). The temperature of heated areas for precipitation hardening stainless steel shall not exceed the aging temperature. Subject to the approval of the engineer of record (EOR), alternative temperature limitations are permitted to be used based on recommendations by the material producer.

2. Thermal Cutting

Modify first paragraph to read as follows:

Thermally cut edges shall meet the requirements of AWS D1.1/D1.1M, clauses 7.14.5.2, 7.14.8.3, and 7.14.8.4 with the exception that thermally cut free edges that

will not be subject to fatigue shall be free of sharp V-shaped notches and gouges greater than 3⁄16 in. (5 mm) in depth. Gouges deeper than 3⁄16 in. (5 mm) and notches shall be removed by grinding or repaired by welding. Notches or gouges deeper than 3⁄16 in. (5 mm) and up to 3⁄8 in. (10 mm) that remain from cutting shall be removed by grinding at a slope not greater than 1:2.5. Notches or gouges 3⁄8 in. (10 mm) deep or greater shall be repaired only with the approval of the EOR. Oxygen gouging is not permitted on quenched and tempered steels.

3. Planing of Edges

Replace section with the following:

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

4. Welded Construction

Replace section with the following:

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

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

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

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

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 welding procedure specification used. The fabricator or constructor, as applicable for the work scope, should develop a method whereby each weld and its associated data can be identified.

7. Dimensional Tolerances

Replace section with the following:

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

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

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

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

(b) Stiffeners
Stiffeners serving as connections shall be located within 1⁄4 in. (6 mm) of the detailed position. A variation of 1 in. (25 mm) is permissible for the location of other stiffeners, except bearing stiffeners, which shall be within one-half of their thickness from the detailed position.

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

(d) Steel-Plate Composite (SC) Structural Elements
Dimensional tolerances of SC structural elements as measured in the fabrication shop shall be as follows:

(1) At tie locations, the perpendicular distance between the opposite faceplates are within plus or minus $t_{sc}/200$, rounded upward to the nearest 1⁄16 in. (2 mm), where t_{sc} is the SC section thickness. This tolerance check shall be performed for the row of ties located closest to the free edges of SC panels.

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

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

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

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

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

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

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

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

Before concrete is placed, the dimensional tolerances for erected modules shall be governed by the erection tolerances defined in the *Code of Standard Practice,* Section 11.3, with the exception that the working lines will be located at one faceplate of the SC structural element.

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

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

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

where

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

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

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

9. Holes for Anchor Rods

Replace section with the following:

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

Add the following new sections:

12. Surface Condition

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

13. Bending

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

14. Commercial Grade Dedication

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

15. Identification of Steel

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

The material shall be identified in one of the following ways as defined by the required use of the material. The material's use shall be defined by the contract documents. If the contract documents do not define the type of identification required, the identification defined in item (a) in the following shall control.

(a) Material identified by grade and size only. Material need only be identified in such a manner that the purchaser is assured that the specified grade is used, and this documentation shall be obtainable throughout the service life of the structure. The fabricator shall maintain the documentation until such time that those documents are transferred to the owner.

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

(c) Material identified by heat number for an individual member, but not subparts, fasteners, or weld consumables. Material test reports shall be identifiable to an individual member in the structure.

(d) Material identified by heat or production lot number to all components of the structure including subparts, fasteners, and weld consumables. Material test reports shall be identifiable to an individual member, subpart, fastener, or weld consumable.

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

NM3. SHOP PAINTING

4. Finished Surfaces

Replace section with the following:

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

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

NM4. ERECTION

2. Stability and Connections

Replace section with the following:

The frame of structural steel buildings and composite steel/concrete structures shall be carried up true and plumb within the limits defined in the *Code of Standard Practice* Section 11 and/or contract documents. Temporary bracing shall be provided in accordance with the requirements of the *Code of Standard Practice* and/or contract documents wherever necessary to support the loads to which the structure is subjected, including equipment and the operation of same. For composite steel/concrete structures, the required bracing shall resist impact and hydrostatic loads of fluid concrete during placement of concrete within the structure. Bracing shall be left in place as long as required for safety.

Add the following new sections:

7. Tolerances for Cranes

7a. Tolerances for Crane Column Base Lines

Crane column base lines shall be established as parallel lines and the column centerlines maintained within 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 documents.

7c. Tolerances for Crane Rails

Center-to-center distances of crane rails and the straightness of crane rails shall meet the tolerances prescribed by "Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes" (CMAA-70). Vertical misalignment of crane rails measured at centerlines of columns shall meet the tolerances prescribed by CMAA-70. For polar cranes, the tolerances in Sections NM4.7a and NM4.7b shall apply, except that the CMAA tolerances for crane span shall be applied for crane rail diameter. Crane rails shall be centered on the crane girders wherever possible. For plate girders and wide-flange shapes (i.e., not box-section beams), in no case shall the real eccentricity be greater than 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 (QC), quality assurance (QA), 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 QC or QA 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 is required to 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 QC or QA for surface preparation or coatings.

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

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

The chapter is organized as follows:

NN1. General Provisions
NN2. Fabricator and Erector Quality Assurance Program
NN3. Fabricator and Erector Documents
NN4. Inspection and Nondestructive Evaluation Personnel
NN5. Minimum Requirements for Inspection of Structural Steel Buildings and Structures
NN6. Minimum Requirements for Inspection of Composite Construction
NN7. Nonconforming Material and Workmanship

NN1. GENERAL PROVISIONS

The fabricator and erector shall include both quality control (QC) and quality assurance (QA) as part of their quality plan as specified in this chapter. When required by the authority having jurisdiction (AHJ), applicable building code (ABC), purchaser, owner, or engineer of record, an independent party shall provide additional oversight to ensure the fabricator and erector are following their QC and QA programs. Nondestructive examination (NDE) shall be performed by an individual, agency, or firm approved by the fabricator or erector responsible for QA.

User Note: The producers of materials manufactured in accordance with standard specifications referenced in Section NA3 and steel deck manufacturers are not considered fabricators or erectors.

NN2. FABRICATOR AND ERECTOR QUALITY ASSURANCE PROGRAM

The fabricator and erector shall establish, maintain, and document procedures and perform inspections to ensure that their work is completed in accordance with the established quality assurance (QA) program, the appropriate elements of the standard, the Nuclear Specification, and the construction documents. The QA program shall be developed in accordance with the ASME standard NQA-1, *Quality Assurance Requirements for Nuclear Facility Applications*, or equivalent.

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

Using the approved fabrication documents, the fabricator's quality assurance inspector (QAI) shall perform inspections of the following as a minimum, as applicable:

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

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

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

(4) Tolerances for shop fabrication in accordance with Section 11 of the *Code of Standard Practice* and Chapter NM

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

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

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

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

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

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

(5) Tolerances for field erection in accordance with Section 11 of the *Code of Standard Practice* and Chapter NM

NN3. FABRICATOR AND ERECTOR DOCUMENTS

1. Submittals for Steel Construction

The fabricator or erector shall submit the following documents in electronic or printed form for review and approval by the owner or the engineer of record (EOR) or their designee in accordance with Section 4 of the *Code of Standard Practice*, prior to fabrication or erection, as applicable:

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

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

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

2. Available Documents for Steel Construction

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

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

(2) For steel castings and forgings, copies of material test reports in accordance with *Specification* Section A3.2.

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

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

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

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

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

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

(9) Welding procedure specifications (WPS).

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

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

(12) Fabricator's or erector's written quality assurance (QA) 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 QA program.

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

2. Quality Assurance Inspector Qualifications

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

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

(b) Qualified under the provisions of AWS D1.1/D1.1M, clause 8.1.4, and AWS D1.6/D1.6M, clause 8, if applicable to stainless steel welding.

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

3. NDE Personnel Qualifications

NDE personnel shall be qualified in accordance with their employer's written practice, which shall meet the criteria of AWS D1.1/D1.1M, clause 8.1.4.2(5), and AWS D1.6/D1.6M, clause 8.1.4.2, if applicable to stainless steel welding, and

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

(b) ASNT CP-189, *Standard for the Qualification and Certification of Nondestructive Testing Personnel*.

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

1. Quality Control

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

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

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

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

2. Quality Assurance

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

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

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

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

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	NA	P
Material identification (type/grade)	NA	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.

NA = not applicable

The quality assurance inspector (QAI) or qualified personnel identified in the QA program shall review the material test reports and certifications as listed in Section NN3.2 for compliance with the construction documents before the fabricated members and components are shipped from the fabricator's plant.

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

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

For QA inspection, the applicable construction documents shall be the approval documents, specifications, and applicable reference codes and standards.

3. Coordinated Inspection

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

TABLE NN5.4-2
Inspection Tasks During Welding

Inspection Tasks During Welding	QC	QA
Use of qualified welders	NA	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

NA = not applicable

4. Inspection of Welding

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

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

User Note: Visual weld acceptance criteria can also be found in the Electric Power Research Institute document NCIG-01, Revision 2, "Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants," NP-5380, Volume 1, September 1987. These nonmandatory inspection guidelines may be used for visual inspection of structural welds made in accordance with the provisions of AWS D1.1/D1.1M if approved by the engineer of record. These guidelines provide background information and instructions to assist the inspector

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.

in evaluating weld attributes. Measuring techniques and guidance on the accuracy, frequency, and locations for measuring welds are discussed. It is important for the inspector to understand weld size tolerance and significant measurements units in order to properly assess the acceptance of each weld.

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

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

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

5. Nondestructive Examination of Welded Joints

5a. Procedures

Ultrasonic testing (UT), magnetic particle testing (MT), penetrant testing (PT), and radiographic testing (RT), where required, shall be performed by qualified NDE personnel in accordance with AWS D1.1/D1.1M and AWS D1.6/D1.6M, as applicable.

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

5b. CJP and PJP Groove Weld NDE

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

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

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

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

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

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

5c. Welded Joints Subjected to Fatigue

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

5d. Increase in Rate of Groove Weld NDE

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

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

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

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

5e. Documentation

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

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

6. Inspection of High-Strength Bolting

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

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

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

TABLE NN5.6-1
Inspection Tasks Prior to Bolting

Inspection Tasks Prior to Bolting	QC	QA
Manufacturer's certifications available for fastener materials	NA	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 2.10)	O	O
NA = not applicable		

TABLE NN5.6-2
Inspection Tasks During Bolting

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

Table NN5.6-3
Inspection Tasks After Bolting

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

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

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

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

Perform (P)—These tasks shall be performed for each bolted connection.

7. Inspection of Galvanized Structural Steel Main Members

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

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

8. Other Inspection Tasks

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

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

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

User Note: This includes such items as braces, stiffeners, member locations, and correct application of joint details at each connection.

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

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

User Note: This includes such items as braces, stiffeners, member locations, and correct application of field joint details at each connection.

NN6. MINIMUM REQUIREMENTS FOR INSPECTION OF COMPOSITE CONSTRUCTION

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

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

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

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

P—Perform these tasks for each steel element.

For welding of faceplates, observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures, and workmanship are in conformance with the construction documents. Steel-plate composite (SC) structural element welding inspection of the module shall include verification of the welding consumables, welding procedure specifications, welding procedure qualification for nonprequalified joints, qualifications of welding personnel prior to the start of the work, observations of the work in progress, and a visual inspection of all completed welds. Tests, materials, and construction requirements for concrete shall comply with the applicable provisions of ACI 349 or ACI 349M. In Tables NN6.2 and NN6.3, the inspection tasks are as follows:

P—Perform these tasks for each steel element.

NN7. NONCONFORMING MATERIAL AND WORKMANSHIP

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

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

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

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

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

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

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

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

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 an inelastic analysis of the associated structure demonstrates that the structure is able to maintain its global stability and structural integrity to withstand all other concurrently acting loads.

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

Add the following as the last paragraph:

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

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

APPENDIX N2

DESIGN OF FILLED COMPOSITE MEMBERS (HIGH STRENGTH)

No changes to Appendix 2 of the Specification.

APPENDIX N3

FATIGUE

No changes to Appendix 3 of the Specification.

APPENDIX N4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

Modify Appendix 4 of the Specification as follows.

N4.1. GENERAL PROVISIONS

Add the following paragraphs after the introductory paragraph:

The intended functions of the structure under a design basis fire shall be stated in the design basis documents. The provisions of Appendix N4 shall be for life safety associated with evacuation of building occupants in the event of a design-basis fire. The Nuclear Specification does not address either "Important to Safety" structural steel members or loading conditions associated with a facility fire.

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

N4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

3a. Thermal Elongation

Replace section with the following:

The coefficients of thermal expansion shall be taken as follows:

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

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

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

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

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

TABLE NA-4.2.2
Properties of Concrete at Elevated Temperatures

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

APPENDIX N5

EVALUATION OF EXISTING STRUCTURES

Replace Appendix 5 of the Specification with the following:

This appendix applies to the evaluation of the strength and stiffness under static loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the engineer of record (EOR) or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section NA3.1. This appendix does not address load testing for the effects of seismic and other dynamic loads. Section N5.4 is only applicable to static vertical gravity loads applied to existing roofs or floors.

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

The appendix is organized as follows:

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

N5.1. GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the design strength of a force resisting member or system. The evaluation shall be performed by structural analysis (Section N5.3), by load tests (Section 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 Sections 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 (Section N5.3) or load tests (Section N5.4). Such properties shall include the yield stress, tensile strength, and percent elongation. Steel grade shall be verified by either certified material test reports (CMTR) or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or ASTM A568/A568M, as applicable. Evidence shall exist that the material used was dedicated and traceability was maintained during fabrication and erection. When steel grade cannot be established by existing documentation, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure to establish the steel properties. Nominal steel properties of steel grades shall be used in the evaluation of existing structures by structural analysis. Use of steel tensile properties greater than nominal values is permissible only when it can be shown that (a) the coupons taken for CMTR or certified report represent the structure being evaluated, and (b) the value selected is derived from a statistical analysis indicating a high confidence level. If necessary, additional coupons from the as-built structure shall be tested to supplement the CMTR or certified report results, as directed by the EOR.

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

3. Chemical Composition

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

4. Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Sections NA3.1d and NA3.1e are critical to the performance of the structure, the Charpy

V-notch toughness shall be determined in accordance with the provisions of Section NA3.1e. If the notch toughness so determined does not meet the provisions of Section NA3.1e, the EOR shall determine if remedial actions are required.

5. Weld Metal

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

6. Bolts

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

N5.3. EVALUATION BY STRUCTURAL ANALYSIS

1. Dimensional Data

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

2. Strength Evaluation

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

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

3. Serviceability Evaluation

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

N5.4. EVALUATION BY LOAD TESTS

1. Determination of Live Load Rating by Testing

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

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

Periodic unloading shall be considered once the service load level is attained and before the load combination $1.2D + 1.6L$ is placed on the structure. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining the maximum test load for one hour, that the deformation of the structure does not increase by more than 10% above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

2. Serviceability Evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. The service test load shall be held for a period of one hour, and deformations shall be recorded at the beginning and at the end of the one-hour holding period.

N5.5. EVALUATION REPORT

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

APPENDIX N6

MEMBER STABILITY BRACING

No changes to Appendix 6 of the Specification.

APPENDIX N7

ALTERNATIVE METHODS OF DESIGN FOR STABILITY

No changes to Appendix 7 of the Specification.

APPENDIX N8

APPROXIMATE ANALYSIS

No changes to Appendix 8 of the Specification.

Add the following Appendix.

APPENDIX N9

STEEL-PLATE COMPOSITE (SC) STRUCTURAL ELEMENTS

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

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

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

The appendix is organized as follows:

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

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

N9.1. DESIGN REQUIREMENTS

1. General Provisions

The following provisions shall apply to SC structural elements**:**

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

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

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

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

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 (345 MPa) nor more than 80 ksi (550 MPa). The minimum elongation shall be at least 15%, and the minimum tensile-to-yield ratio, F_u/F_y, shall be 1.20.

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

Lightweight concrete shall not be used.

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

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

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

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

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

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

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

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

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

2. Design Basis

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

2a. Required Strength

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

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

2b. Design for Stability

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

3. Faceplate Slenderness Requirement

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

(a) For connection regions

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

(b) For interior regions

$$\frac{b}{t_p} \leq 1.20\sqrt{\frac{E_s}{F_y}} \tag{A-N9-2b}$$

where

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

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

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

t_p = thickness of faceplate, in. (mm)

4. Requirements for Composite Action

4a. Classification of Shear Connectors

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

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

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

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

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

4b. Spacing of Shear Connectors

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

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

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

where

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

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

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

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

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

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

$$s \le c_1 \sqrt{\frac{Q_{cv}^{avg} l t_{sc}}{\dfrac{M_n}{2.5 t_{sc}}}} \tag{A-N9-4}$$

where

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

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

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

5. Tie Requirements

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

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

5a. Classification of Ties

Ties shall be classified as yielding shear reinforcement when

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

where

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

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

Otherwise, ties shall be classified as nonyielding shear reinforcement.

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

5b. Tie Spacing

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

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

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

where

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

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

$$\alpha = 1.7\left[\frac{t_{sc}}{t_p} - 2\right]\left[\frac{t_p}{D_{tie}}\right]^4 \tag{A-N9-7}$$

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

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

6. Design and Detailing Requirements for Impactive and Impulsive Loads

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

7. Design and Detailing Requirements for Openings

User Note: Faceplate holes for reinforcing steel dowels or for other types of joining instruments that have a diameter less than 2½ in. (63 mm) and $t_{sc}/8$ do not constitute as openings.

7a. Design and Detailing Requirements for Small Openings

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

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

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

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

(1) Analysis is permitted to be performed without modeling the opening provided that the panel section where the opening is located shall be evaluated considering 25% reduction in all available strengths. Alternatively, the effect of a small opening shall be accounted for by conducting an analysis that meets the Section N9.1.7b(a) requirements (1) and (2).

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

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

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

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

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

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

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

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

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

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

7b. Design and Detailing Requirements for Large Openings

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

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

(1) The size of the opening modeled for analysis purposes shall be larger than the physical opening such that it extends to where the faceplates are fully developed away from the boundary of the opening.

(2) No reductions shall be applied to the available strengths of the panel sections in the vicinity of the as-modeled opening.

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

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

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

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

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

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

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

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

(a) The region affected by a concentrated bank of small openings shall be treated as a large opening when the smallest clear distance between adjacent small openings is between t_{sc} and $2t_{sc}$ for the interior region and $0.5t_{sc}$ and $1.5t_{sc}$ for the connection region.

(b) The bank of small openings shall be reinforced using a single reinforcing plate that:

(1) incorporates all sleeves within the bank of openings;

(2) meets the requirements of Section N9.1.7a(b); and

(3) extends the minimum required distance beyond the perimeter of the outermost openings.

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

N9.2. ANALYSIS REQUIREMENTS

1. General Provisions

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

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

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

(b) Second-order effects shall be addressed in accordance with Section N9.1.2b.

(c) Finite element analyses involving accident thermal conditions shall be conducted in accordance with Section N9.2.4.

(d) The viscous damping ratio for safe shutdown earthquake (SSE) level seismic analysis shall not exceed 5% for the determination of required strengths for SC structural elements.

2. Effective Stiffness for Analysis

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

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

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

where

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

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

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

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

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

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

t_c = concrete infill thickness, in. (mm)

w_c = weight of concrete per unit volume $\left(90 \leq w_c \leq 155 \text{ lb/ft}^3 \text{ or } 1\,500 \leq w_c \leq 2\,500 \text{ kg/m}^3\right)$

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

ρ' = stiffness-adjusted reinforcement ratio
= ρn

n = modular ratio of steel and concrete
= E_s/E_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) When $S_{rxy} \le S_{cr}$

$$\begin{aligned}(GA)_{eff} &= (GA)_{uncr} \\ &= G_sA_s + G_cA_c\end{aligned} \tag{A-N9-9}$$

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)

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

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

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

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

$$= \frac{0.063\sqrt{f'_c}}{G_c}(GA)_{uncr} \tag{A-N9-10}$$

$$= \frac{0.17\sqrt{f'_c}}{G_c}(GA)_{uncr} \tag{A-N9-10M}$$

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

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

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

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

where

$$(GA)_{cr} = 0.5\bar{\rho}^{-0.42}G_sA_s \tag{A-N9-12}$$

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

$$= \frac{A_sF_y}{31.6A_c\sqrt{f'_c}} \tag{A-N9-13}$$

$$= \frac{A_sF_y}{83A_c\sqrt{f'_c}} \tag{A-N9-13M}$$

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

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

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

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

3. Geometric and Material Properties for Finite Element Analysis

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

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

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

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

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

4. Analyses Involving Normal Operating and Accident Thermal Conditions

4a. Requirements for Normal Operating Thermal Conditions

For normal operation or other long-term period exposure:

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

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

4b. Requirements for Accident Thermal Conditions

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

Higher steel surface temperatures than those provided in this section are permitted if reduction in strength determined by testing or other rational criteria is applied to

design. In addition, the engineer of record shall justify, by testing or other rational criteria, that increased temperatures do not cause deterioration of SC structural elements with or without the postulated loads.

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

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

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

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

where

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

α_s = thermal expansion coefficient of faceplate, °F^{-1} (°C^{-1})

ΔT_{sg} = maximum temperature difference between faceplates due to accident thermal conditions, °F (°C)

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

5. Determination of Required Strengths

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

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

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

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

N9.3. DESIGN OF SC STRUCTURAL ELEMENTS

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

1. Uniaxial Tensile Strength

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

2. Compressive Strength

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

The following symbols and definitions are used in addition to or as replacements of those used in the Specification Section I2.1b:

P_{no} = nominal compressive strength per unit width, kip/ft (N/m)
$$= F_y A_{sn} + 0.85 f_c' A_c \quad \text{(A-N9-16)}$$
P_e = elastic critical buckling load per unit width, kip/ft (N/m)
$$= \pi^2 \left(EI\right)_{eff} \Big/ L_c^2 \quad \text{(A-N9-17)}$$
A_c = area of the concrete infill per unit width, in.^{2}/ft (mm^2/m)
$= lt_c$, in.^{2}/ft (mm^2/m)
A_{sn} = net area of faceplates per unit width, in.^{2}/ft (mm^2/m)
E_c = modulus of elasticity of concrete
$= w_c^{1.5}\sqrt{f_c'}$, ksi $\left(0.043 w_c^{1.5}\sqrt{f_c'}\text{, MPa}\right)$
$(EI)_{eff}$ = effective SC stiffness per unit width for buckling evaluation, kip-in.^{2}/ft (N-mm^2/m)
$$= E_s I_s + 0.60\, E_c I_c \quad \text{(A-N9-18)}$$
I_c = moment of inertia of concrete infill per unit width
$= lt_c^3/12$, in.^{4}/ft (mm^4/m)

I_s = moment of inertia per unit width of faceplates (corresponding to the condition when concrete is fully cracked)
$= l\left[t_p\left(t_{sc}-t_p\right)^2/2\right]$, in.^{4}/ft (mm^4/m)
L_c = effective length of member, in. (mm)
l = 12 in./ft (1000 mm/m)

3. Out-of-Plane Flexural Strength

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

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

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

where

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

4. In-Plane Shear Strength

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

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

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

where

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

$$K_s = G_s A_s \tag{A-N9-21}$$

$$K_{sc} = \frac{0.7\left(E_c A_c\right)\left(E_s A_s\right)}{4E_s A_s + E_c A_c} \tag{A-N9-22}$$

V_{ni} = nominal in-plane shear strength per unit width of SC panel section, kip/ft (N/m)
l = 12 in./ft (1000 mm/m)

5. Out-of-Plane Shear Strength

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

(1) Project-specific large-scale out-of-plane shear tests

(2) Test results

(3) The provisions of this section

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

ϕ_{vo} (LRFD) = 0.90 for SC panel sections with yielding ties, except when Section N9.3.5(b) applies and V_{conc} exceeds V_s
= 0.75 for all other cases

Ω_{vo} (ASD) = 1.67 for SC panel sections with yielding ties, except when Section N9.3.5(b) applies and V_{conc} exceeds V_s
= 2.00 for all other cases

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

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

$$V_{no} = V_{conc} + V_s \tag{A-N9-23}$$

where

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

$$= 0.063(f'_c)^{0.5}t_c l \tag{A-N9-24}$$

$$= 0.166(f'_c)^{0.5}t_c l \tag{A-N9-24M}$$

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

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

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

$$= \xi\left(\frac{t_c}{s_{tl}}\right)F_t\left(\frac{l}{s_{tt}}\right) \tag{A-N9-25}$$

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

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)

ξ = 1.0 for yielding shear reinforcement
= 0.5 for nonyielding shear reinforcement

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

(b) The nominal out-of-plane shear strength per unit width of SC panels, V_{no}, with shear reinforcement spaced greater than half the section thickness shall be the greater of V_{conc} and V_s. V_{conc} shall be calculated using Equation A-N9-24 or Equation A-N9-24M, and V_s shall be calculated using Equation A-N9-25, taking both ξ and $\left(t_c/s_{tl}\right)$ as 1.0.

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

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

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

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

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

(1) For nonyielding shear reinforcement:

$$\left[\left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_x + \left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_y\right]^{5/3} + \left[\frac{\sqrt{V_{rx}^2 + V_{ry}^2}\Big/\left(0.9t_{sc}\right)}{\Psi\left(lQ_{cv}^{avg}/s^2\right)}\right]^{5/3} \le 1.0 \tag{A-N9-26a}$$

(2) For yielding shear reinforcement:

$$\left[\left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_x + \left(\frac{V_r - V_{c\ conc}}{V_c - V_{c\ conc}}\right)_y\right]^{2} + \left[\frac{\sqrt{V_{rx}^2 + V_{ry}^2}\Big/\left(0.9t_{sc}\right)}{\Psi\left(lQ_{cv}^{avg}/s^2\right)}\right]^{2} \le 1.0 \tag{A-N9-26b}$$

where

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

V_c = available out-of-plane shear strengths per unit width of SC panel section in local x $\left(V_{cx}\right)$ and y $\left(V_{cy}\right)$ 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 $\left(V_{rx}\right)$ and y $\left(V_{ry}\right)$ directions using LRFD or ASD load combinations, kip/ft (N/m)

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

s = spacing of shear connectors, in. (mm)

x = subscript relating symbol to the local x-axis
y = subscript relating symbol to the local y-axis
Ψ = 1.0 for panel sections with yielding shear connectors
= 0.5 for panel sections with nonyielding shear connectors

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

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

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

V_c = V_{no}/Ω_{vo}, kip/ft (N/m), where V_{no} is calculated in accordance with Section N9.3.5
$V_{c\ conc}$ = V_{conc}/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 Equations A-N9-26a and A-N9-26b.

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-N9-27 to A-N9-29. These equations shall be used with the maximum and minimum required principal in-plane strengths per unit width for the notional half of the SC panel section, $S_{r,max}$ and $S_{r,min}$, calculated using Equations A-N9-30 to A-N9-33.

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

$$\alpha\left(\frac{S_{r,max}+S_{r,min}}{2V_{ci}}\right)+\left(\frac{S_{r,max}-S_{r,min}}{2V_{ci}}\right)\leq 1.0 \qquad \text{(A-N9-27)}$$

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

$$\frac{S_{r,max}}{V_{ci}}-\beta\left(\frac{S_{r,max}+S_{r,min}}{V_{ci}}\right)\leq 1.0 \qquad \text{(A-N9-28)}$$

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

$$-\beta\left(\frac{S_{r,min}}{V_{ci}}\right) \leq 1.0 \qquad \text{(A-N9-29)}$$

where

$$S_{r,max}, S_{r,min} = \frac{S'_{rx} + S'_{ry}}{2} \pm \sqrt{\left(\frac{S'_{rx} - S'_{ry}}{2}\right)^2 + \left(S'_{rxy}\right)^2} \qquad \text{(A-N9-30)}$$

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

$$= \frac{S_{rx}}{2} \pm \frac{M_{rx}}{j_x t_{sc}} \qquad \text{(A-N9-31)}$$

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

$$= \frac{S_{ry}}{2} \pm \frac{M_{ry}}{j_y t_{sc}} \qquad \text{(A-N9-32)}$$

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

$$= \frac{S_{rxy}}{2} \pm \frac{M_{rxy}}{j_{xy} t_{sc}} \qquad \text{(A-N9-33)}$$

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

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

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

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

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

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

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

= 0.67

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

α = V_{ci}/T_{ci}

β = V_{ci}/P_{ci}

Alternatively, for each notional half, the interaction shall be limited directly with the required in-plane membrane strengths per unit width (S'_{rx}, S'_{ry}, S'_{rxy}), using Equations A-N9-34 to A-N9-36. S'_{rx}, S'_{ry}, and S'_{rxy} shall be calculated using Equations A-N9-31 to A-N9-33.

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

$$\left(1-\alpha^2\right)\left(\frac{S'_{rx} + S'_{ry}}{2V_{ci}}\right)^2 + \alpha\left(\frac{S'_{rx} + S'_{ry}}{V_{ci}}\right) + \left[\frac{\left(S'_{rxy}\right)^2 - S'_{rx}S'_{ry}}{V_{ci}^2}\right] \leq 1.0 \qquad \text{(A-N9-34)}$$

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

$$\beta(1-\beta)\left(\frac{S'_{rx}+S'_{ry}}{V_{ci}}\right)^2 + (1-2\beta)\left(\frac{S'_{rx}+S'_{ry}}{V_{ci}}\right) + \left[\frac{\left(S'_{rxy}\right)^2 - S'_{rx}S'_{ry}}{V_{ci}^2}\right] \leq 1.0 \quad \text{(A-N9-35)}$$

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

$$\beta^2\left[\frac{\left(S'_{rxy}\right)^2 - S'_{rx}S'_{ry}}{V_{ci}^2}\right] - \beta\left(\frac{S'_{rx}+S'_{ry}}{V_{ci}}\right) \leq 1.0 \quad \text{(A-N9-36)}$$

where

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

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

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

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

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

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

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

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

$\phi_{ci} = 0.80$

$\phi_{ti} = 1.00$

$\phi_{vs} = 0.95$

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

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

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

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

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

$\Omega_{ci} = 1.88$

$\Omega_{ti} = 1.50$

$\Omega_{vs} = 1.58$

User Note: Use of the alternative interaction equations, A-N9-34, A-N9-35, and A-N9-36, may result in total interaction values that are negative; such instances should be interpreted as the element having satisfied the applicable interaction equation.

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

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

N9.4. DESIGN OF SC STRUCTURAL ELEMENT CONNECTIONS

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

User Note: Examples of such connections include the following:

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

1. General Provisions

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

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

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

1a. Required Strength

The required strength for the connections shall be determined as:

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

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

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

1b. Available Strength

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

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

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

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

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

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

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

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

2. Lap Splicing of Reinforcing Bars with Faceplates

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

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

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

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

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

Add the following Appendix.

APPENDIX N10

SPECIAL PROVISIONS FOR IMPACTIVE AND IMPULSIVE LOADS

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

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

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

The appendix is organized as follows:

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

N10.1. GENERAL PROVISIONS

1. Additional Material Requirements

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

(a) The specification of the material of those structures or structural elements that are subjected to impactive and/or impulsive loads shall comply with Section NA3.1. Welds subject to impactive and/or impulsive loads shall comply with Section NJ2.6.

(b) Bolts and threaded parts shall be in accordance with Section NJ3.14.

(c) The structural documents and specifications shall meet Section NA4.

2. Dynamic Strength Increase

It is permitted to consider strain rate-adjusted material strengths for structural steel, reinforcing steel, and concrete materials. The material strength increase shall be

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

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

NA = not applicable

based on applicable experimental data. The Dynamic Increase Factors (DIF) specified in Table A-N10.1.1 are permitted for use in the absence of experimental data.

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

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

3. Load Effects and Load Combinations

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

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

1. Compactness Requirements

For structural steel and composite members subjected to flexure or compression due to impactive and impulsive load, the width-to-thickness ratios of their compression elements shall not exceed the limiting width-to-thickness ratio, λ_c, provided in Table A-N10.2.1. The R_y values necessary for determination of λ_c in Table A-N10.2.1 shall be obtained from Table A3.2 in the *Seismic Provisions*, where R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y, of that material.

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

2. Local Response Evaluation

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

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

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

3. Special Design and Detailing Requirements for Ductility

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

4. Analysis Requirements for Verification of Structural Element Ductility

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

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

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

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

$$\mu_r = \frac{D_m}{D_y} \tag{A-N10-1}$$

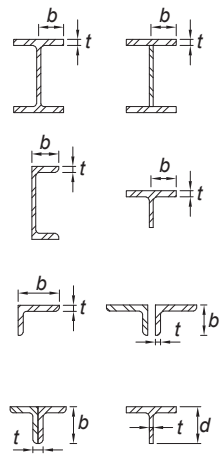
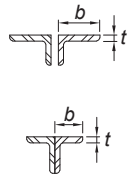
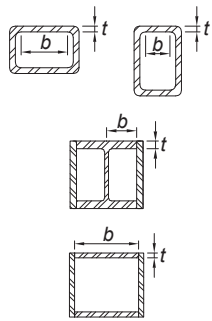
where

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

D_y = effective yield deflection, in. (mm)

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

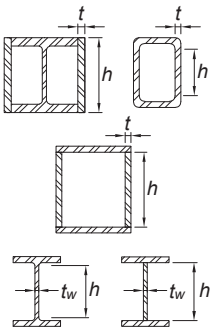
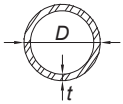
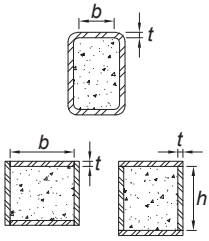
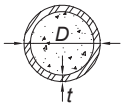
TABLE A-N10.2.1
Limiting Width-to-Thickness Ratios for Structural Steel and Composite Members

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

D = Outside diameter of round HSS, in. (mm); E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa); d = full depth of the section, for stems of tees, in. (mm); t_w = thickness of web, in. (mm)

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

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

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

D = Outside diameter of round HSS, in. (mm); E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa); d = full depth of the section, for stems of tees, in. (mm); t_w = thickness of web, in. (mm)

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

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

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

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

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

ε_y = strain corresponding to nominal yield stress = F_y/E

[c]Accompanying compression force, if any, shall be less than the smaller of $0.1F_eA_g$ and $0.1F_yA_g$, where A_g is the gross area of member, in.^{2} (mm^2), and F_e is the elastic buckling stress, ksi (MPa).

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

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

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

The method per option (b) is easier to implement because it involves a simplified bilinear (or multilinear) resistance function of the structural element's load-displacement behavior that is based on an equivalent single-degree-of-freedom model (accordingly, the permissible ductility ratios in Table A-N10.2.2 have been conservatively specified). This method is based on similar provisions in UFC 3-340-03 (DOD, 2008), which requires determination of

the structural element's resistance function by using its nominal yield strength times the dynamic increase factor and applicable strain-hardening effect. As defined and illustrated in UFC 3-340-02 (DOD, 2008), the effective yield point is taken as the intersection point of the line representing the initial equivalent stiffness with the horizontal line representing the plastic behavior (see commentary for further discussion). The associated effective yield displacement is used for implementation of option (b).

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

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

1. Compactness Requirements

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

2. Local Response Evaluation

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

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

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

where

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

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

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

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

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

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

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

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

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

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

K_p = strength-dependent concrete penetrability factor

$$= 5.692\Big/\sqrt{f_c'} \tag{A-N10-5}$$

$$= 14.95\Big/\sqrt{f_c'} \tag{A-N10-5M}$$

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

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

N = missile nose shape factor per the modified NDRC formula
= 0.72 for flat-nosed missiles
= 0.84 for blunt-nosed missiles
= 1.00 for spherical-nosed missiles
= 1.14 for sharp-nosed missiles

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

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

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

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

W_p = missile weight, lb (N)

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

f'_c = compressive strength of concrete, ksi (MPa)

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

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

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

r_2 = concrete frustum radius at the inside face of the back faceplate, in. (mm)

$$= r_1 + \left(t_c - x_{sc}\right)\tan\theta \text{ when } x_{sc} < t_c \quad \text{(A-N10-8a)}$$

$$= 0 \text{ when } x_{sc} \geq t_c \quad \text{(A-N10-8b)}$$

t_c = concrete infill thickness, in. (mm)

x_c = concrete penetration depth for the reinforced concrete section of the same thickness as the SC cross section, in. (mm)

$$= \sqrt{4K_p N W_p d\left(\frac{V_i}{1{,}000d}\right)^{1.80}} \text{ when } \frac{x_c}{d} \leq 2.0 \quad \text{(A-N10-9a)}$$

$$= K_p N W_p \left(\frac{V_i}{1{,}000d}\right)^{1.80} + d \text{ when } \frac{x_c}{d} > 2.0 \quad \text{(A-N10-9b)}$$

$$= 0.511\sqrt{K_p N W_p d\left(\frac{V_i}{d}\right)^{1.80}} \text{ when } \frac{x_c}{d} \leq 2.0 \quad \text{(A-N10-9aM)}$$

$$= 0.0652 K_p N W_p \left(\frac{V_i}{d}\right)^{1.80} + d \text{ when } \frac{x_c}{d} > 2.0 \quad \text{(A-N10-9bM)}$$

x_{sc} = missile penetration depth into the SC cross section, in. (mm)

$$= K_{psc}\, x_c \quad \text{(A-N10-10)}$$

α_p = missile deformability factor per NEI 07-13

= 0.60 for deformable missiles

= 1.00 for rigid missiles

θ = inclination angle of the concrete frustum extending from the penetration depth of the impacting missile to the back faceplate of the impacted SC section, degrees

$$= \frac{45°}{\left(t_c/d\right)^{1/3}} \quad \text{(A-N10-11)}$$

ρ_c = concrete density, lb/ft^3 (kg/m^3)

σ_r = equivalent radial compressive stress in the rear faceplate, based on von Mises yield criterion, ksi (MPa)

$= 5.1F_y + 101$ when $t_p \geq 0.25$ in. (A-N10-12a)

$= 3.9F_y + 64$ when $t_p < 0.25$ in. (A-N10-12b)

$= 5.1F_y + 696$ when $t_p \geq 6$ mm (A-N10-12aM)

$= 3.9F_y + 441$ when $t_p < 6$ mm (A-N10-12bM)

3. Special Analysis, Design, and Detailing Requirements

Ductility shall be verified in accordance with Section N10.3.4(c) when either of the following conditions is present:

(a) Large opening(s)

(b) Small opening(s) with free edge at the opening parameter

User Note: Where possible and practical, an independent impact barrier structure should be provided to spare an SC structural element with either condition (a) or (b) from being directly subjected to impulsive or impactive loads.

Bolted attachments to the tension faceplate are permitted if the net section fracture limit state does not control. Except for the case of shop welding associated with the reinforcement around a small opening, welded attachments to the tension faceplate are not permitted in regions that are expected to undergo yielding when subjected to the specified impulsive or impactive loading.

Only yielding shear reinforcement is permitted. Additionally, the available out-of-plane shear strength shall be at least 120% of the out-of-plane shear strength corresponding to the flexure-controlled failure mechanism.

User Note: The out-of-plane shear strength requirement specified in this section ensures that the SC structural element subjected to impactive or impulsive load will undergo significant inelastic response through flexural yielding, rather than the significantly less ductile failure mechanism associated with the yielding of shear reinforcement.

4. Analysis Requirements for Verification of Structural Element Ductility

The response of SC structural elements subjected to impactive and impulsive loads shall be determined by one of the following three methods:

(a) If the target response remains elastic, the dynamic load effects of the impulsive or impactive loads shall be calculated using the applicable dynamic load factor (DLF). The calculated maximum elastic demands using this method shall not exceed the capacities defined in Appendix N9, Section N9.3.

(b) If the target response is in the inelastic range, use of a simplified single-degree-of-freedom analysis of the target, using either bilinear or multi-linear resistance function, is permitted. The presence of concurrent membrane forces, if any, shall be accounted for when developing the resistance function. The calculated maximum support rotation using this method shall not exceed 6 degrees (0.105 rad).

(c) Alternatively, if the target response is in the inelastic range, use of a detailed nonlinear and inelastic finite element analysis is permitted for direct determination of plastic strains. The maximum plastic strain using this method shall not exceed 0.05 in./in. (mm/mm) for the faceplates and 0.005 in./in. (mm/mm) for ties classified as yielding shear reinforcement.

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

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

SC basemats subjected to impulsive or impactive loads shall be evaluated using option (c).

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

COMMENTARY

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

October 4, 2024

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 360-22).

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.

COMMENTARY SYMBOLS

The Commentary uses the following symbols in addition to the symbols defined in the Nuclear Specification. The section number in the righthand column refers to the Commentary section where the symbol is first used.

Symbol	Definition	Section
A_j	Effective cross-sectional area within joint, in.^{2} (mm^2)	App. N9.4.1b
E'_c	Effective concrete compression stiffness, ksi (MPa)	App. N9.3.1
$(EI)_{cr\text{-}tr}$	Cracked-transformed flexural stiffness of the SC section, kip-in.^{2} (N-mm^2)	App. N9.2.2
F_{steel}	Compression force carried by the faceplates, kips (N)	NM2.7
$F_{y0.2}(T)$	Specified minimum yield stress of steel at elevated temperature using the 0.2% strain offset method, ksi (MPa)	Table C-NB3.1
G	Shear modulus of elasticity, ksi (MPa)	App. N9
K	Effective length factor	App. N9.1.3
K_{Ic}	Critical stress intensity factor for static loading and plane-stress conditions of maximum constraint	NA.3.1
K_{Id}	Critical stress intensity factor for dynamic (impact) loading and plane-stress conditions of maximum constraint	NA.3.1
K_s	Contribution of faceplates to in-plane shear stiffness, kip/in. (N/mm)	App. N9.2.2
K_{sc}	Contribution of cracked orthotropic concrete to in-plane shear stiffness, kip/in. (N/mm)	App. N9.2.2
K_{xy}^{cr}	Cracked stiffness, kip/in. (N/mm)	App. N9.2.2
K_{xy}^{sec}	Secant stiffness, kip/in. (N/mm)	App. N9.2.2
K_{xy}^{uncr}	Uncracked stiffness, kip/in. (N/mm)	App. N9.2.2
L_{TR}	Transfer length, in. (mm)	App. N9.1.5
L_v	Length of the composite wall subjected to out-of-plane shear loading, in. (mm)	App. N9.1.4b
M	Out-of-plane bending moment, kip-in. (N-mm)	App. N9.1.4b
M_{th}	Thermal moment, kip-in. (N-mm)	App. N9.2.4
Q_{cv}	Design shear strength of connector, kip (N)	App. N9.1.4a
Q_{cv}^{tie}	Available interfacial shear strength of the tie bars, kips (N)	App. N9.3.6a
R_t	Ratio of the expected tensile strength to the minimum specified tensile strength	App. N9.1.5a
S_{sc}	Concrete cracking shear strength, kips (N)	App. N9.2.2
S_{rxy}	Applied shear force, kips/in. (N/mm)	App. N9.2.2
S_{rxy}^{Y}	In-plane shear force, kips/in. (N/mm)	App. N9.3.4
S_{xy}^{cr}	Shear strength at concrete cracking, kips/in. (N/mm)	App. N9.3.1
S_{xy}^{y}	Yield shear strength, kips/in. (N/mm)	App. N9.3.1
V	Out-of-plane shear, kips (N)	App. N9.1.4b
V_i	Initial velocity, ft/s (m/s)	App. N10.3.2
V_{nu}	Ultimate in-plane shear strength, kip/in. (N/mm)	App. N9.3.4
V_p	Perforation velocity, ft/s (m/s)	App. N10.3.2

Symbol	Definition	Section
c	Distance to the neutral axis, in. (mm)	App. N9.2.2
c_c	Depth of the triangular concrete compressive stress block, in. (mm)	App. N9.3.3
e	Top plate strain	App. N9.2.2
f_{cy}	Concrete compressive stress at the onset of faceplate yielding for pure in-plane shear loading, ksi (MPa)	App. N9.1.1
k_E, $k_{y0.2}$	Retention factors for steel	NB3.1
k_p	Proportional limit retention factor	NB3.1
k_y	Yield stress retention factor	NB3.1
n	Concrete-to-steel modular ratio	App. N9.2.2
n_{es}	Effective number of steel anchors contributing to a unit cell	App. N9.3.6a
n_{et}	Effective number of ties contributing to a unit cell	App. N9.3.6a
s_L	Longitudinal spacing of steel anchors, in. (mm)	App. N9.1.4b
s_T	Transverse spacing of steel anchors, in. (mm)	App. N9.1.4b
t_{sc}	Fabricated panel thickness, in. (mm)	NM2.7
ΔT_{s1}^{max}, ΔT_{s2}^{max}	Maximum surface temperature increases on the two faceplates, °F (°C)	App. N9.2.2
γ	Parameter for joint shear strength	App. N9.4.1b
ε_{cr}	Buckling strain	App. N9.1.3
ε_{sh}	Shrinkage strain	App. N9.2.2
ε_y	Yield strain	App. N9.1.3
λ_p	Limiting width-to-thickness ratio (compact/noncompact)	App. N10.2
μ	Ductility factor	App. N10.2
ν	Poisson's ratio of steel	App. N9.2.2
ν_c	Poisson's ratio of concrete infill	App. N9.3.1
ν_s	Poisson's ratio of steel faceplates	App. N9.3.1
ρ'	Stiffness-normalized reinforcement ratio	App. N9.2.2
ρ_t	Shear reinforcement ratio	App. N9.4.1b
$\sigma_{c\text{-}p2}$	Concrete minimum principal compressive stress, ksi (MPa)	App. N9.1.1
ϕ_{th}	Thermal curvature, in.$^{-1}$ (mm^{-1})	App. N9.2.4

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 2022 AISC *Specification for Structural Steel Buildings* (AISC, 2022a), 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, 2023).

The 2022 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2022b), 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 2022 AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2022c), 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 2024 edition of the Nuclear Specification includes a revision to the Charpy V-notch (CVN) testing temperature for structures or structural components that are subject to impactive and/or impulsive loads. The previous requirement of testing the materials at a temperature that is at least 30°F (17°C) below the lowest anticipated service temperature of the structural component being evaluated had existed since the first edition of the Nuclear Specification in 1984. That previous requirement was based on historical information used by the ASME *Boiler and Pressure Vessel Code* (ASME, 2023).

Since the issuance of the first edition of the Nuclear Specification, research has demonstrated that the CVN testing temperature was conservative.

In updating this requirement, the operating environmental temperature of the nuclear facility when a potential impactive/impulsive accident were to occur was considered. The external temperature of the nuclear facility seldom is below 0°F (–18°C); tornadic-driven missiles at this external temperature were also unlikely to occur. The temperature 0°F (–18°C) was selected conservatively as the lowest potential temperature condition for which the structural component would experience this impactive/impulsive accident.

In addition to the lowest potential temperature condition, one of the most important critical design parameters for CVN fracture toughness determination is the time for the target (i.e., structure, component, or support) to be stressed to their maximum load carrying capacity (e.g., yielding or buckling.) For convenience, this time was described as the loading response time. Experience indicates that loading response time is somewhat variable, but the smallest time would be critical. Values of 1 second, 0.1 second, and 0.05 seconds were considered; accordingly, 0.05 seconds was selected for further study.

After establishing this lowest potential temperature condition and the loading response time, the temperature for CVN testing was re-evaluated; the corresponding CVN energy values for Table NA3.1 were likewise revised.

The minimum CVN fracture toughness was determined using established empirical correlations between the critical stress intensity factor for dynamic (impact) loading and plane-stress conditions of maximum constraint, K_{Id}, and CVN ft-lb, and a temperature shift equation between K_{Id} and the critical stress intensity factor for static loading and plane-stress conditions of maximum constraint, K_{Ic} (Barsom and Rolfe, 1999). The same methodology (Barsom, 1975) has been previously applied to establish CVN requirements for statically designed buildings, for buildings subjected to seismic loads, for steel bridge structures, and for weld metals associated with these types of structures.

The CVN fracture toughness correlations and the related methodology that was used to establish minimum CVN requirements incorporate several conservative assumptions. For example, the correlation equations predict values that are lower than those exhibited by the experimental data. The CVN temperature requirement is 50°F (10°C) above the transition temperature, and the CVN impact values are increased beyond the 15 ft-lb (20 J) used to define the temperature shift. The required CVN ft-lb value and test temperature are for fracture critical conditions where failure of a single component in the structure results in collapse of the entire structure. Also, they are established for steel with a yield strength of 65 ksi (450 MPa) and are conservatively applied to 50-ksi (345-MPa) steel and 36-ksi (250-MPa) steel. The correlations are based on fracture of a single steel plate and are conservatively applied to a steel-plate composite (i.e., Appendix N9).

Applying this methodology to fracture critical conditions for 65-ksi (450-MPa) steel to zero minimum operating temperature, and for 0.05 second loading response time, the CVN requirement for the average of three specimens is: 25 ft-lb (34 J) at 0°F (–18°C). The corresponding minimum CVN value of 20 ft-lb (27 J) for any one of the three specimens is based on ASTM standards.

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. Examples of components that are subjected to impactive and/or impulsive loads are jet shields, pipe whip restraints, and pipe whip impact barriers. Impactive loads include the following examples: tornado-borne missiles, whipping pipes, aircraft impact, and other internal and external missiles. Impulsive loads include the following examples: jet impingement loads, blast pressure, compartment pressurization, and jet shield reactions.

1a. Listed Materials

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 structural element 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.

AISC has recently published ANSI/AISC 370-21, *Specification for Structural Stainless Steel Buildings* (AISC, 2021) and AISC Design Guide 27, *Structural Stainless Steel*, 2nd.Ed. (Baddoo and Meza, 2022). Designers of nuclear facilities are encouraged to use the appropriate sections of these documents until such time that ANSI/AISC N690 addresses structural stainless steel.

1d. Rolled Heavy Shapes

Modify this section as follows:

1d. Rolled Heavy Shapes

and

1e. 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 l984 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).

5. Consumables for Welding

Add the following:

Because nuclear facilities sometimes utilize stainless steel structural materials, AWS D1.6/D1.6M (AWS, 2017), AWS A5.4/A5.4M (AWS, 2022a), and AWS A5.9/A5.9M (AWS, 2022b) is included in 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 DOCUMENTS AND SPECIFICATIONS

Add the following:

Because of the stringent requirements for quality control and inspection in nuclear facilities, the additional requirements for structural design documents and specifications provided in Section NA4.1 are necessary.

Add the following section:

NA6. QUALITY ASSURANCE

This section is included 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, 2022), 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 349-13 or ACI 349M-13 (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 7-22 (ASCE, 2022), 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.

Section NB2.1 states that the snow load, S, is as stipulated in ASCE/SEI 7-22 for Risk Category IV facilities. Risk Category IV facilities are defined in Table 1.5-1 of ASCE/SEI 7-22 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 7-22 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.

2. Severe Environmental Loads

Similar to snow load, S, wind load, W, is also stipulated in ASCE/SEI 7-22 for Risk Category IV facilities. The importance factor for wind loads has been deleted (from previous editions of ASCE/SEI 7) due to changes in new wind hazard maps.

The operating basis earthquake (OBE) is generally not used in the design unless specifically selected by the owner/license applicant as design input.

4. Abnormal Loads

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

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

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

5. Load and Resistance Factor Design (LRFD)

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

The load combinations stem from a probability-based study of load combinations for design of nuclear power plants (Hwang et al., 1987). The probabilistic methodology in that study is consistent with that used to develop the probability-based load combination requirements appearing in ASCE/SEI 7-22 (ASCE, 2022), 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 Chapter 26 of ASCE/SEI 7-22 (ASCE, 2022). This wind load addresses extreme nontornadic wind effects from extratropical storms and hurricanes. Tornadic wind effects are defined by W_t, and are addressed in Load Combination NB2-7 for extreme environmental effects. The extreme environmental loads, W_t and E_s, as specified in NUREG-0800 (NRC, 2007) and in 10 CFR Part 50 (Code of Federal Regulations, 2020), are design-basis events and thus appear in the load combinations with load factors of unity.

There is often a time evolution (and spatial variation) associated with abnormal load effects such as pipe whip impact, jet impingement, compartment pressurization, accident thermal, etc. Accordingly, there is a possibility that only some of them (i.e., without the accompaniment of the others) may turn out to be most critical for specific structures or structural elements. On the other hand, it could sometimes be too conservative (and impractical) to postulate all of them to be occurring simultaneously. This consideration may also affect how the safe shutdown earthquake load is combined with the abnormal loads.

Regarding the tornado load effects, the maximum depressurization intensity manifests when the eye of the tornado is passing over the top of an above-ground structure. In contrast, the occurrence of maximum intensities of tornado velocity pressure and the prospect of a tornado missile impact precedes the instant when the depressurization load effect is at its maximum. Accordingly, the three tornado load effects can be combined in accordance with the treatment provided in Section 3.3.2 of NRC NUREG 0800.

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 has shown 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 include 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 a rational method that establishes material properties at elevated temperatures) 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 a rigorous second-order analysis should be used for code checks.

In all cases, only differential settlements that produce significant structural effects need to be accounted for.

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 7-22 (ASCE, 2022) 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 NB2.5 for additional discussion, including thermally induced member forces and moments and situations when one or more loads may not be acting concurrently.

NB3. DESIGN BASIS

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

***Add the following paragraphs*:**

The strength of steel decreases at elevated temperatures. Where the structural component or system is exposed to sustained temperatures in excess of 250°F (120°C), the decrease should be taken into account in determining the design strength. Design values for steel strength at elevated temperatures may be obtained from any reference that is based on validated test data.

Although minor reductions to carbon steel material properties may occur prior to the 250°F (120°C) limit, these are typically small and need not be directly considered. Caution should be practiced when utilizing retention factors for material properties of carbon steel due to sustained elevated temperatures, particularly the values tabulated in *Specification* Appendix 4 (AISC, 2022a). The values presented for the yield stress retention factor, k_y, correspond to 2% strain. This definition of yield contains significant nonlinearity in the material model (as demonstrated by the significant difference between the proportional limit retention factor, k_p, and k_y) that is not

acceptable for service level design. This is especially true for limit states involving buckling and instabilities, which have been developed based on linear elastic-perfectly plastic stress-strain relationships. Limiting the design to the proportional limit would be overly conservative in many situations. One potential alternative is the use of the 0.2% offset method with the material models from *Specification* Appendix 4. Table C-NB3.1 provides such retention factors for a 36-ksi steel and a 50-ksi steel along with the proportional limit retention factors. These values compare well with published data for a variety of steels [ASME Code Section II Part D and ASME Code Case N-71-20 (ASME, 2023), Lee, J. et al. (2013), and Lee and Choi (2021)] and would be acceptable for shear and tension limits states including plastic moment calculations. However, nonlinearities between k_p and $k_{y0.2}$ still exist, particularly at and above 400°F (200°C). Agarwal and Varma (2011) present a method to address buckling and instability limit states by using the 0.2% offset to define yield along with further reductions to the modulus of elasticity to create equivalent elastic perfectly plastic stress-strain curves. The effective modulus of elasticity is determined by maintaining a consistent area underneath the stress-strain curves between the nonlinear material model and the simplified elastic perfectly plastic model. While Agarwal and Varma (2011) focused on ASTM A992/A992M structural steel (i.e., 50-ksi yield), Table C-NB3.1 also presents modulus of elasticity retention factors for a 36-ksi structural steel.

Limited data is available for sustained loads on Group 120 and Group 150 bolts while subjected to elevated temperatures above 600°F (320°C). For bolted connections subjected to sustained elevated temperatures above 600°F (320°C), ASTM A193/A193M Grade B7 bolts should be considered.

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 (120°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 any reference that is based on validated test data. See Commentary NB3.1 for additional information.

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 interaction.

TABLE C-NB3.1
Properties of Carbon Steel at Elevated Temperatures

Steel Temperature, °F (°C)	*Specification* Appendix 4		$F_y = 36$ ksi		$F_y = 50$ ksi	
	$k_E = E(T)/E = G(T)/G$	$k_P = F_P(T)/F_y$	$k_E = E(T)/E = G(T)/G$	$k_{y0.2} = F_{y0.2}(T)/F_y$	$k_E = E(T)/E = G(T)/G$	$k_{y0.2} = F_{y0.2}(T)/F_y$
68 (20)	1.00	1.00	1.00	1.00	1.00	1.00
200 (93)	1.00	1.00	1.00	1.00	1.00	1.00
400 (200)	0.90	0.80	0.81	0.89	0.83	0.89
600 (320)	0.78	0.58	0.62	0.77	0.66	0.78
700 (370)	0.72	0.47	0.53	0.72	0.58	0.72

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* (Hucek, 1985) and in the ASME *Boiler and Pressure Vessel Code*, Section II, Part D, Material Properties (ASME, 2023). Sustained temperature above 700°F (370°C) may make the material sensitive to creep effects that need to be considered in the design. It should be noted that the reduced material properties for structural steel in Appendix N4 are established for fire conditions and not appropriate for the design of structural steel for elevated temperature service. See Commentary NB3.1 for for related information.

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 (120°C), the effect of this decrease on structural stiffness and deformations should be taken into account.

Add the following section:

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

Refer to the Commentary for Appendix N10.

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 a rational method that establishes material properties at elevated temperatures.

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 318-19 or ACI 318M-19 (ACI, 2019), have been included.

CHAPTER NJ

DESIGN OF CONNECTIONS

Modify Chapter J of the Specification Commentary as follows.

NJ2. WELDS AND WELDED JOINTS

6. Filler Metal Requirements

Add the following:

Additional notch toughness requirements are incorporated into this section. The provisions are based on the *Seismic Provisions* (AISC, 2022b).

NJ3. BOLTS, THREADED PARTS, AND BOLTED CONNECTIONS

Add the following:

14. 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 7-22 (ASCE, 2022) 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, 2017) is part of the the Nuclear Specification.

The provisions of ASME *Boiler and Vessel Code*, Section III (ASME, 2023), are applicable at the weld interface of steel-plate composite (SC) structural 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 structural element panels and sub-modules fabricated in the shop and inspected before release

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

(c) Erected SC structural element modules before concrete casting

(d) Constructed SC structures after concrete casting

SC structural element panels are typically fabricated in the shop and then shipped to the field. The overall dimensions of the fabricated SC structural element panels are limited by the applicable shipping restrictions. SC structural element 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. Additionally, SC structural element 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 subject to the same size restrictions as the wall panels.

SC structural element 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 structural element 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 structural element 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, 2020) or AWS D1.6/D1.6M (AWS, 2017), 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/D1.1M for carbon steel and AWS D1.6/D1.6M 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* (AISC, 2022c). 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 structural elements with nonslender and slender faceplates. Finite element models of nonslender SC structural elements with faceplate waviness (imperfections) up to $0.65t_p$ were analyzed. The faceplates developed more than 95% of their yield strength (i.e.,

TABLE C-NM2.1
Thickness Tolerances for Fabricated SC Structural Element Panels and Sub-Modules

Wall Thickness, t_{sc}, in. (mm)	Wall Panel Thickness Tolerance at Tie Locations, in. (mm)	Wall Panel Thickness Tolerance Between Tie Locations, in. (mm)
24 (610)	±⅛ (±3)	±¼ (±6)
30 (760)	±3⁄16 (±5)	±5⁄16 (±8)
36 (910)	±3⁄16 (±5)	±⅜ (±10)
42 (1100)	±¼ (±6)	±7⁄16 (±11)
48 (1200)	±¼ (±6)	±½ (±13)
54 (1400)	±5⁄16 (±8)	±9⁄16 (±14)
60 (1500)	±5⁄16 (±8)	±⅝ (±16)

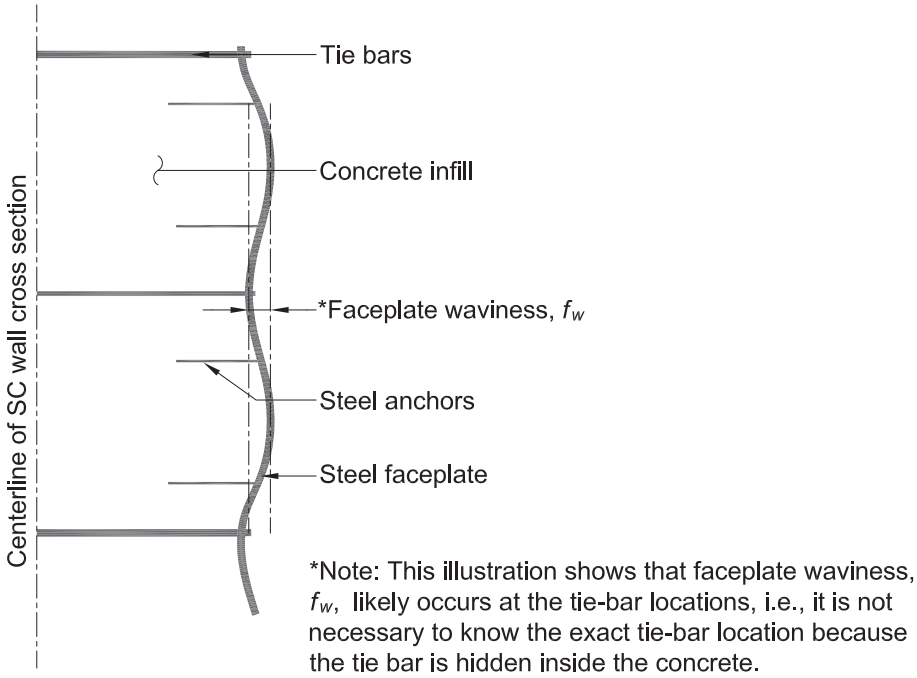
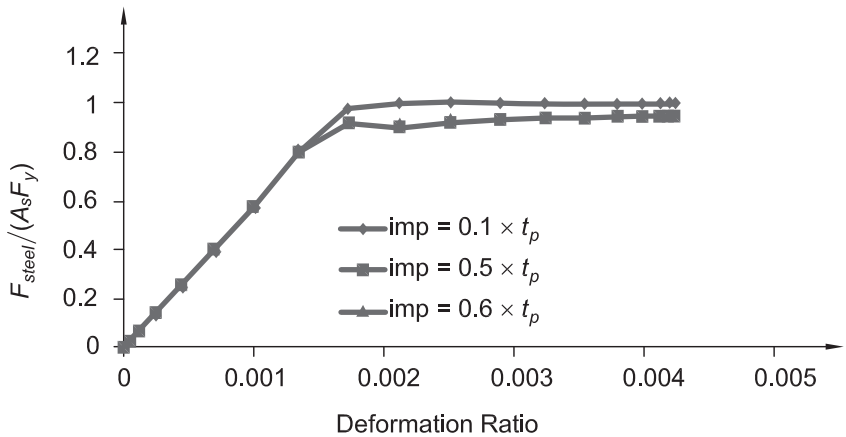
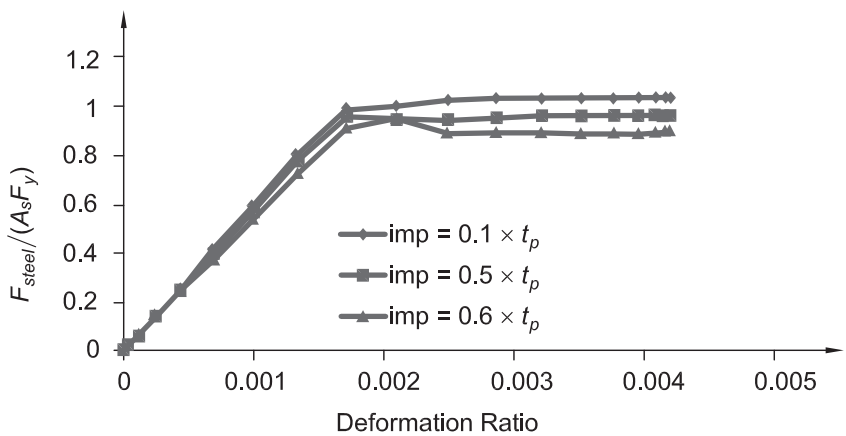


Fig. C-NM2.1. Faceplate waviness. (The faceplate waviness and the variation in tie-bar dimensions has been exaggerated for illustration purposes.)

$0.95A_sF_y$) at the axial compressive strength. Figure C-NM2.2 was developed using the results of the finite element analyses. It illustrates the compression force carried by the faceplates, F_{steel}, normalized with respect to its nominal 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.



(a) $s/t_p = 24$, $t_p = 0.25$ *in.*



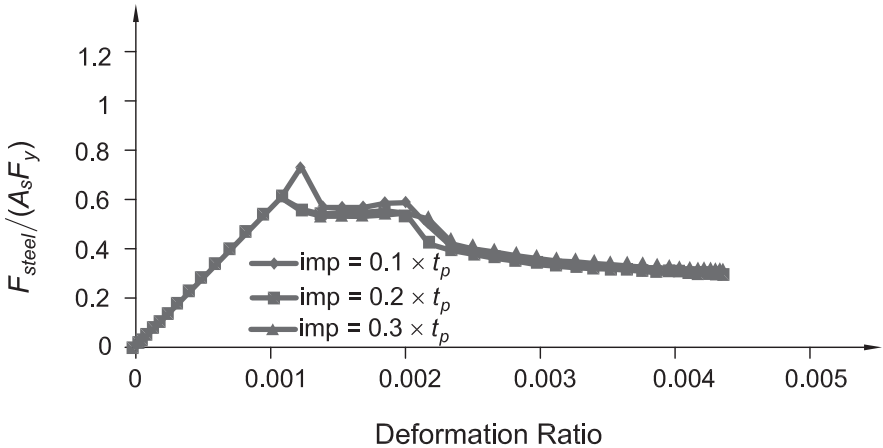
(b) $s/t_p = 24$, $t_p = 0.50$ *in.*

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

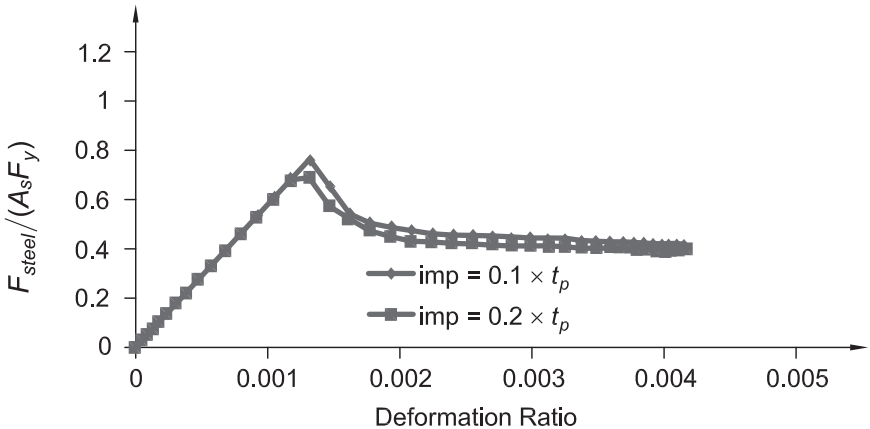
NM3. SHOP PAINTING

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.



(c) $s/t_p = 36$, $t_p = 0.25$ *in.*



(d) $s/t_p = 36$, $t_p = 0.50$ *in.*

Fig. C-NM2.2 (cont'd). Normalized force carried by faceplates versus average strain.

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 structural element panel, sub-module, or module before it is placed in the erected position. The tolerances for the SC structural element are inspected in the fabrication shop and in the erected condition. Because the SC structural element 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, 2020). 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.

CHAPTER NN

QUALITY CONTROL AND QUALITY ASSURANCE

Replace Chapter N of the Specification Commentary with the following:

Because of the unique quality assurance (QA) requirements applicable to nuclear facilities, the fabricator's QA and quality control (QC) 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, 2022) stipulates the requirements for the establishment and execution of QA programs for nuclear facilities. QA 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, 2022) establishes installation, inspection, and testing requirements for various structural items, including structural steel.

The Nuclear Specification's usage of the terms QA and QC differ from the *Specification*. A QA 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. QC 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 QA and QC are applied to fabricated structural steel. In both the *Specification* and the Nuclear Specification, the QC functions are performed by the fabricator or erector. In the *Specification* the QA functions are performed by the authority having jurisdiction (AHJ), applicable building code, purchaser, owner, or engineer of record (EOR). In the Nuclear Specification, the QA functions are performed by the fabricator or the erector as defined in their quality program. The fabricator's or erector's quality program is 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 QA 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

5. Nondestructive Examination of Welded Joints

The intent of the nondestructive examination requirements of this section is to confirm that the contractor has a program to be able to place quality welds. The intent is not to test the ability of an individual welder.

5b. CJP and PJP Groove Weld NDE

The 10% sampling approach was 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.

5e. Documentation

Usage of NCIG-01 (EPRI, 1987a), NCIG-02 (EPRI, 1987b) and NCIG-03 (EPRI, 1987c) 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 structural elements accordingly. Mock-ups may be employed to demonstrate that a particular construction technique provides adequate quality of concrete placement in SC structural elements.

Honeycombing 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 structural elements.

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, 2022a). 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.

Add the following Appendix to the Specification Commentary.

APPENDIX N9

STEEL-PLATE COMPOSITE (SC) STRUCTURAL ELEMENTS

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 structural elements are plate or shell-type structures; they are typically not part of frame structures. In SC construction, concrete walls are reinforced with faceplates connected to each other using steel ties. The faceplates may also be anchored to concrete using steel anchors if needed. The behavior of SC structural elements 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 structural elements.

This appendix provides specifications for SC structural elements 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 structural elements.

This appendix permits the use of stainless steel materials, but the provisions need to be applied judiciously to stainless steel SC structural elements. The modulus of elasticity and shear modulus of elasticity values for stainless steel are based on the values taken from ASME *Boiler and Pressure Vessel Code*, Section II: Materials—Part D: Properties (Customary) (ASME, 2023), 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 structural elements 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 structural elements satisfying the general requirements of Section N9.1.1. The faceplates of the SC structural elements 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 structural elements with only two faceplates on the exterior surfaces and no additional reinforcing bars. SC structural elements 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.

Composite plate shear walls covered in Chapter NI are similar to steel-plate composite (SC) walls; however, the Chapter NI requirements are meant for flexure-controlled standalone shear walls. In contrast, the provisions of this appendix are intended for the typical labyrinthine and gravity load-carrying squat shear walls that are encountered in nuclear facilities.

The flowchart in 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) structural elements needs to be consistent with the intended behavior of the overall structure and the assumptions made in their analysis.

1. General Provisions

(a) The specified minimum thickness values are based on practical fabrication considerations and the minimum required penetration resistance for tornado missile impact (in case of exterior walls and roof slabs).

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. Similar to reinforced concrete, there is no limitation to wall thickness. Large- and full-scale tests have been performed, and detailed finite element benchmarking analyses have been carried out to understand the behaviors that can be extrapolated to thicker walls. Therefore, it was determined that no upper limit is necessary for the thickness of an SC structural element as long as the general requirements in Section N9.1.1 are satisfied. There is no upper limit for reinforced concrete (RC) element thickness in ACI 349-13 and ACI 349-13M (ACI, 2013), and general requirements for SC structural elements are at least as robust as those for RC elements.

(b) Typically, at least 0.25-in.- (6-mm-) thick 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). 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 0.10) 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 structural elements for in-plane shear loading. [See additional relevant commentary discussion under Item (e).]

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 structural elements with 36 in. (900 mm) concrete thickness, f'_c = 6 ksi (40 MPa), and faceplates with F_y = 50 ksi (345 MPa). As shown, the concrete minimum principal compressive stress $\left(\sigma_{c\text{-}p2}\right)$ changes from $-0.15 f'_c$ to $-0.46 f'_c$ as the reinforcement ratio increases from 0.015 to 0.10. The upper limit of 0.10 for reinforcement ratio is based on this in-plane shear behavior and additional experimental data for very high reinforcement ratios. [See additional relevant commentary discussion under Item (e) for the effect of F_y values, especially when the reinforcement ratios is also larger than 0.05.]

(d) A minimum yield stress of 50 ksi (345 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 structural elements. 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/A36M steel. Additionally, high-strength steels with yield stress greater than 80 ksi (550 MPa) are not permitted because they are typically less ductile, and hence, not desirable for beyond-safe shutdown earthquake shaking.

(e) The use of concrete with specified compressive 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 specified so that the minimum principal (compressive) stress in concrete remains in the elastic range while faceplate yielding occurs under in-plane shear loading.

This edition of the Nuclear Specification incorporates two changes that impact the requirement in (e)—it increases the permissible reinforcement ratio to 0.10 [Item (c)] and permits up to 80-ksi steel grades for use as steel faceplates [Item (d)]. If the design engineer inadvertently selects a somewhat low concrete strength mix for SC applications featuring high reinforcement ratio and/or high faceplate yield strength, then it creates a possibility that the concrete stress at the onset of faceplate yielding under pure in-plane shear loading will be too high. This consideration warrants that the minimum concrete compressive strength be predicated on the faceplate yield strength as well as the reinforcement ratio. This is accomplished as follows.

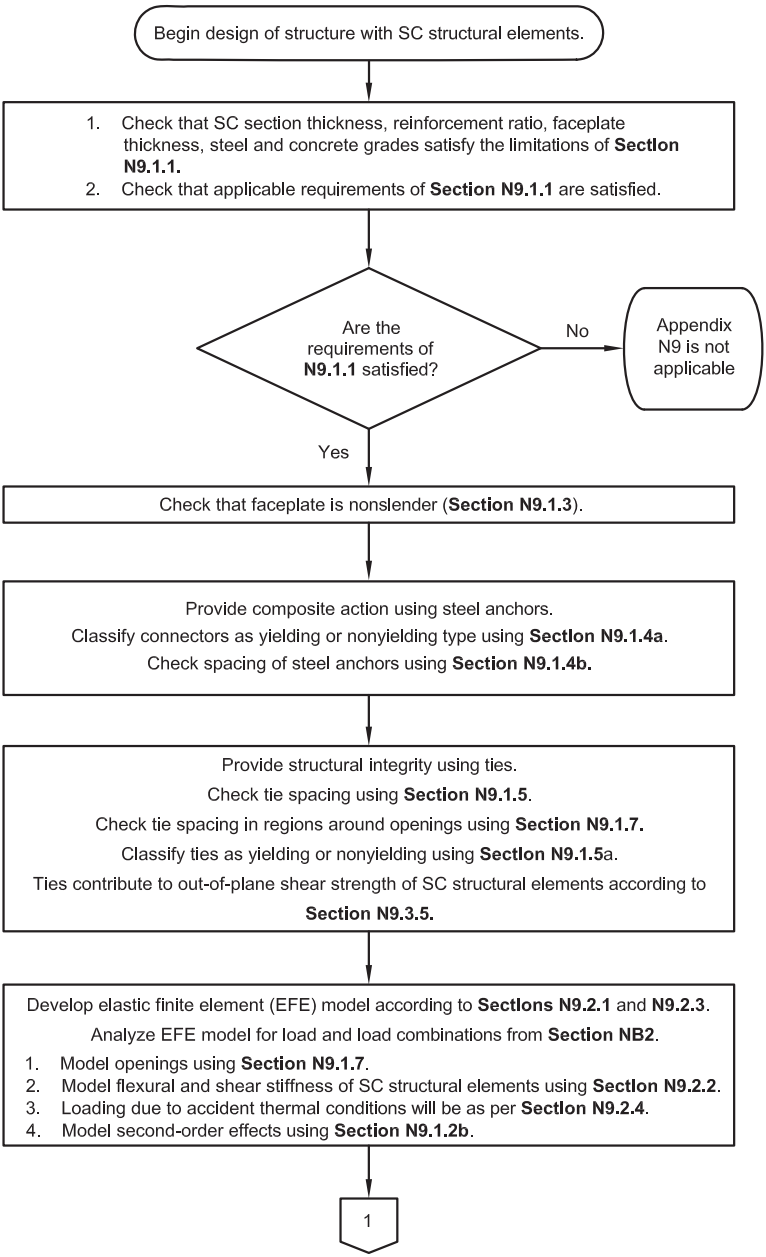
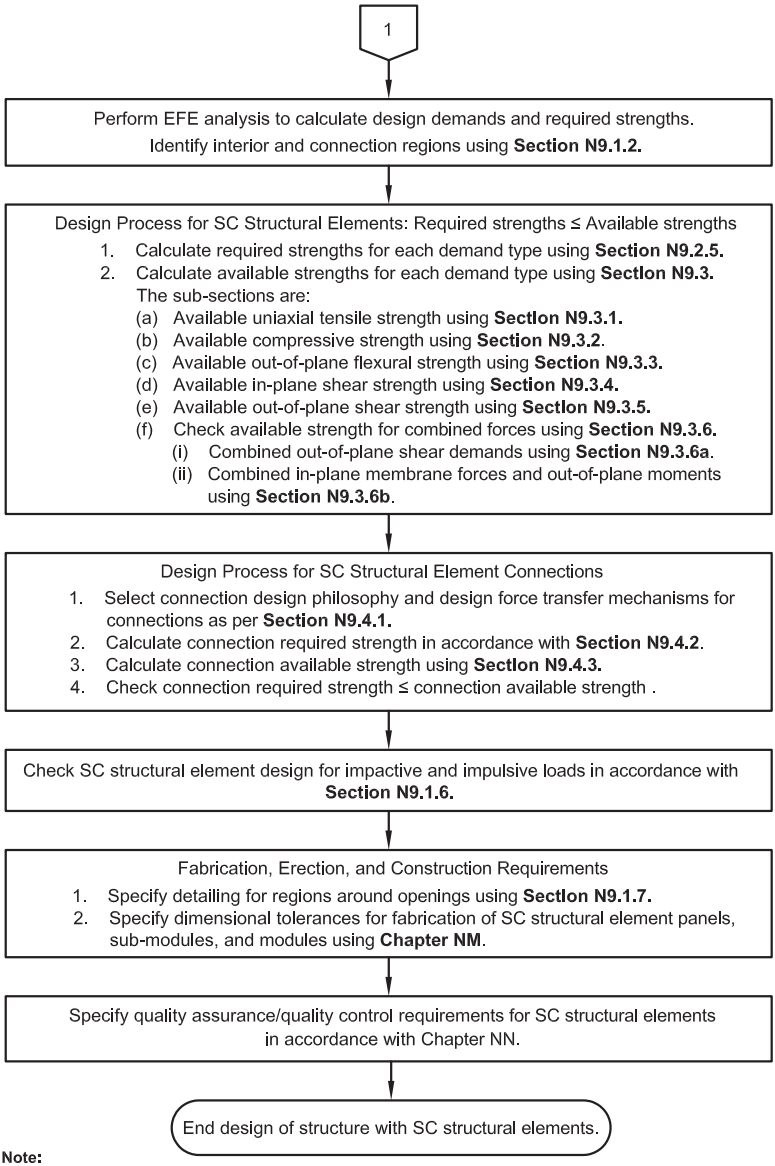


Fig. C-A-N9.1.1. Flowchart to facilitate use of Appendix N9.



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 flexural strengths are in kip-in./ft (N-mm/m).
(b) The required and available membrane in-plane axial 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 (cont'd). Flowchart to facilitate use of Appendix N9.

TABLE C-A-N9.1-1
Principal Stresses Due to In-Plane Shear Loading at Yield Load

Reinforcement Ratio	Faceplate 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.)	$\sigma_{s\text{-}p1}$ (Max.)	$\sigma_{s\text{-}p2}$ (Min.)
	in. (mm)	in. (mm)	kip/in. (kN/mm)	ksi (MPa)	ksi (MPa)	ksi (MPa)	ksi (MPa)
0.015	0.27 (6.86)	36.00 (914.4)	28.78 (5.04)	53.30 (367.49)	7.44 (51.30)	0.00 (0.00)	–0.91 (–6.27)
0.020	0.36 (9.14)	36.00 (914.4)	37.65 (6.59)	52.29 (360.53)	4.95 (34.13)	0.00 (0.00)	–1.14 (–7.86)
0.030	0.54 (13.72)	36.00 (914.4)	54.39 (9.52)	50.36 (347.22)	0.73 (5.03)	0.00 (0.00)	–1.53 (–10.55)
0.040	0.72 (18.29)	36.00 (914.4)	70.01 (12.26)	48.62 (335.22)	–2.66 (–18.34)	0.00 (0.00)	–1.84 (–12.67)
0.050	0.90 (22.86)	36.00 (914.4)	84.72 (14.84)	47.07 (324.54)	–5.42 (–37.37)	0.00 (0.00)	–2.08 (–14.34)
0.060	1.08 (27.43)	36.00 (914.4)	98.73 (17.29)	45.71 (315.16)	–7.69 (–53.02)	0.00 (0.00)	–2.28 (–15.72)
0.070	1.26 (32.00)	36.00 (914.4)	112.17 (19.64)	44.51 (306.86)	–9.59 (–66.12)	0.00 (0.00)	–2.44 (–16.82)
0.080	1.44 (36.58)	36.00 (914.4)	125.16 (21.92)	43.46 (299.65)	–11.19 (–77.16)	0.00 (0.00)	–2.58 (–17.79)
0.090	1.62 (41.15)	36.00 (914.4)	137.79 (24.13)	42.53 (293.23)	–12.55 (–86.53)	0.00 (0.00)	–2.70 (–18.62)
0.100	1.80 (45.72)	36.00 (914.4)	150.12 (26.29)	41.70 (287.51)	–13.73 (–94.67)	0.00 (0.00)	–2.80 (–19.31)

It is generally desirable to limit the concurrent concrete compressive stress to below $0.5f'_c$ because this helps assure that the SC structural element will have additional reserve in-plane shear strength beyond the limit state of faceplate yielding. (See the discussion on ultimate in-plane shear strength and equation in Commentary N9.3.4.) Such an approach in turn also helps ensure that, for SC structural elements subjected to combined in-plane and out-of-plane forces, the actual failure envelop associated with the concrete crushing limit state will remain comfortably beyond the conservative interaction envelop shown in Figure C-A-N9.3.9 and its associated interaction equations specified in Appendix N9, Section N9.3.6b.

The concrete compressive stress at the onset of faceplate yielding for pure in-plane shear loading is obtained from the following equation:

$$f_{cy} = \left(\frac{-S_{xy}^{cr}\left(\nu_c + 1\right)\left(\nu_s + 1\right)}{2t_p E_s\left(\nu_c + 1\right) + E_c t_{sc}\left(\nu_s + 1\right)} - \frac{\left(\nu_s + 1\right)\left(S_{xy}^{y} - S_{xy}^{cr}\right)}{2t_p E_s + t_{sc} E_c'} \right) E_c' \quad \text{(C-A-N9-1)}$$

where

S_{xy}^{cr} = shear strength at concrete cracking, kip/in. (N/mm)
S_{xy}^{y} = yield shear strength, kip/in. (N/mm)
E_c = secant elastic modulus of the concrete infill, ksi (MPa)
E_c' = effective concrete compression stiffness, ksi (MPa)
= $0.7E_c$
E_s = elastic modulus of the steel faceplates, ksi (MPa)
f_c' = compressive strength of the concrete infill, ksi (MPa)
t_p = thickness of faceplate, in. (mm)
t_{sc} = section thickness, in. (mm)
ν_c = Poisson's ratio of the concrete infill
ν_s = Poisson's ratio of the steel faceplates

Figure C-A-N9.1.2 shows the relationship between f_c'/F_y and reinforcement ratio, ρ, subject to the constraint of $f_{cy} < 0.5f_c'$. Based on the actual curve, a straight-line relationship was developed to specify the minimum f_c' value as a function of F_y and ρ—this relationship has been included in the provisions as a minimum concrete strength requirement. It is seen from the figure that f_c'/F_y is about 0.12 when the reinforcement ratio is 0.10. This means that the minimum concrete compressive strength will need to be at least 9.6 ksi when 80-ksi faceplates and 10% reinforcement ratio are used. Based on this, it is concluded that concrete with 10-ksi minimum compressive strength will be satisfactory for the most taxing design combination of 10% reinforcement ratio and faceplates with 80-ksi yield stress.

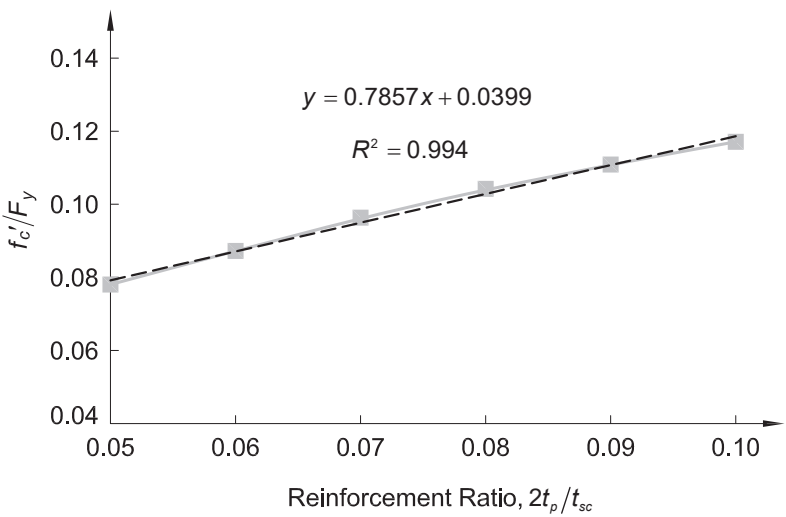


Fig. C-A-N9.1.2. Relationship between f_c'/F_y *and reinforcement ratio* ρ.

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-N9.1.3 presents the range of concrete compressive strength from the experimental database for out-of-plane shear tests. The figure is based on the dataset discussed in Sener and Varma (2014). Concrete specimens with 10 ksi or higher concrete strength were used for testing of composite plate shear walls per Chapter NI. As such, an upper limit of 10 ksi is specified for concrete compressive strength.

This standard subscribes to ACI 349-13 and ACI 349-13M (ACI, 2013) for concrete materials and concrete mix design related issues. Accordingly, concrete materials/mix design related aspects are not addressed in this standard.

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

(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.

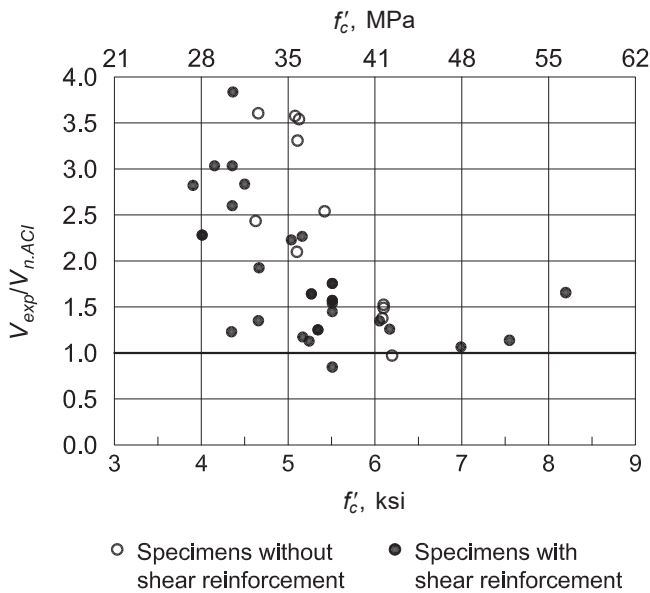


Fig. C-A-N9.1.3. Range of concrete compressive strength from experimental database (Sener and Varma, 2014).

(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 1.10 times 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 structural elements 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 Section N9.1.5 consider delamination due to 50% nonuniformity between the faceplate yield strengths (thickness × yield stress). However, 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 structural elements 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 structural elements, these steel ribs should be welded to the faceplates to fully develop the yield strength of their connected element (leg). The ribs behave as enablers of composite action, and hence, it is necessary to develop their yield strength just like in the case of headed stud anchors. Also, a premature fracture of the rib welds due to inadequate weld strength could jeopardize the faceplate integrity, which is to be avoided.

As shown in Figure C-A-N9.1.4, 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 structural element 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 determination of available strengths.

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

Vent Holes. The faceplates of SC structural elements 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, slabs, and foundations used in liquid or water storage tanks, where the faceplates may be in direct contact with hot water during accident conditions.

Curved SC Elements. The appendix was developed for straight SC structural elements. If the SC elements have any curvature, the effects of curvature on detailing and design need to be evaluated. This is necessary as there is no specific data available for curved SC structural elements 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 structural elements are warranted.

Alternate design methods for SC structural elements 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)].

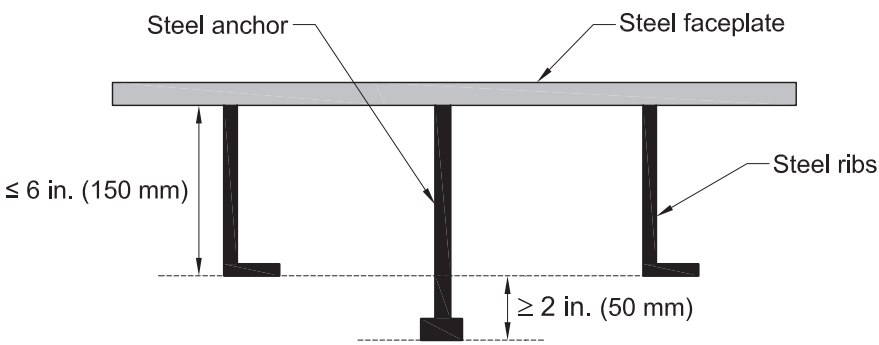


Fig. C-A-N9.1.4. Embedment depth of steel ribs.

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 and slabs is notionally divided into interior regions and connection regions. Force transfer between SC sections, and composite action between faceplates and concrete, develops over connection regions. Figure C-A-N9.1.5 illustrates the typical interior and connection regions for SC structural elements.

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 Commentary N10.3.4, 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 it is comparable to ACI 349-13 or ACI 349M-13, Appendix F (ACI, 2013).

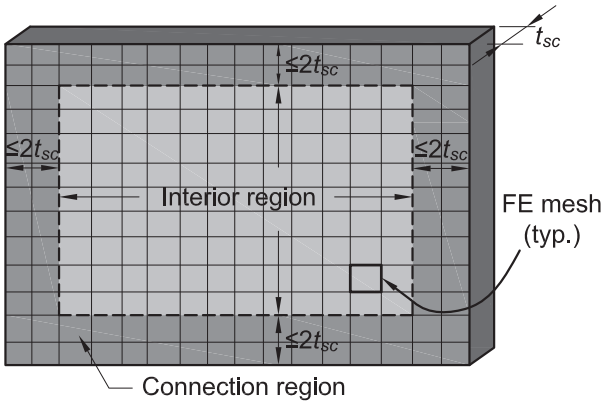


Fig. C-A-N9.1.5. The expanse of SC structural elements separated into connection regions and interior regions.

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 structural elements in nuclear applications will generally exceed 2 ft (0.6 m). Their typical height-to-thickness ratios will meet the requirements of ACI 318-19 or ACI 318M-19, Section 6.2.5(b) (ACI, 2019). Second-order analysis will generally be unnecessary for the labyrinthine structures where SC structural elements will be used. In the rare situation that the ACI requirements are not satisfied, the structure will generally meet the limitations of *Specification* Appendix 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 reinforced concrete walls.

Only vertical SC structural elements are scrutinized for the need to perform second-order analyses. This is because an SC structure supports the sustained gravity loads primarily through (nominally) vertical walls and possibly some columns. Additionally, the slabs experience little, if any, sustained membrane forces. The slab membrane forces, if present, pose negligible second-order effects because their magnitudes are typically a fraction of their buckling load capacities.

Second-order analysis is not warranted in most cases because the typical vertical SC structural elements 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 318-19 or ACI 318M-19 (ACI, 2019), Equations 6.2.5.1a, 6.2.5.1b, and 6.2.5.1c, and therefore, P-δ effects are negligible as well. In rare situations where the requirements of ACI 318-19 and ACI 318M-19, Section 6.2.5.1(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

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-N9.1.6. 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, 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-N9.1.7 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 $\left(s/t_p \times F_y/E\right)$. As shown, ε_{cr} is reasonably consistent with Euler's curve with a partially fixed (effective length factor, $K = 0.7$) 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. The requirement to ensure yielding prior to local buckling is critical for the connection regions. For interior regions, where ductility is not as critical

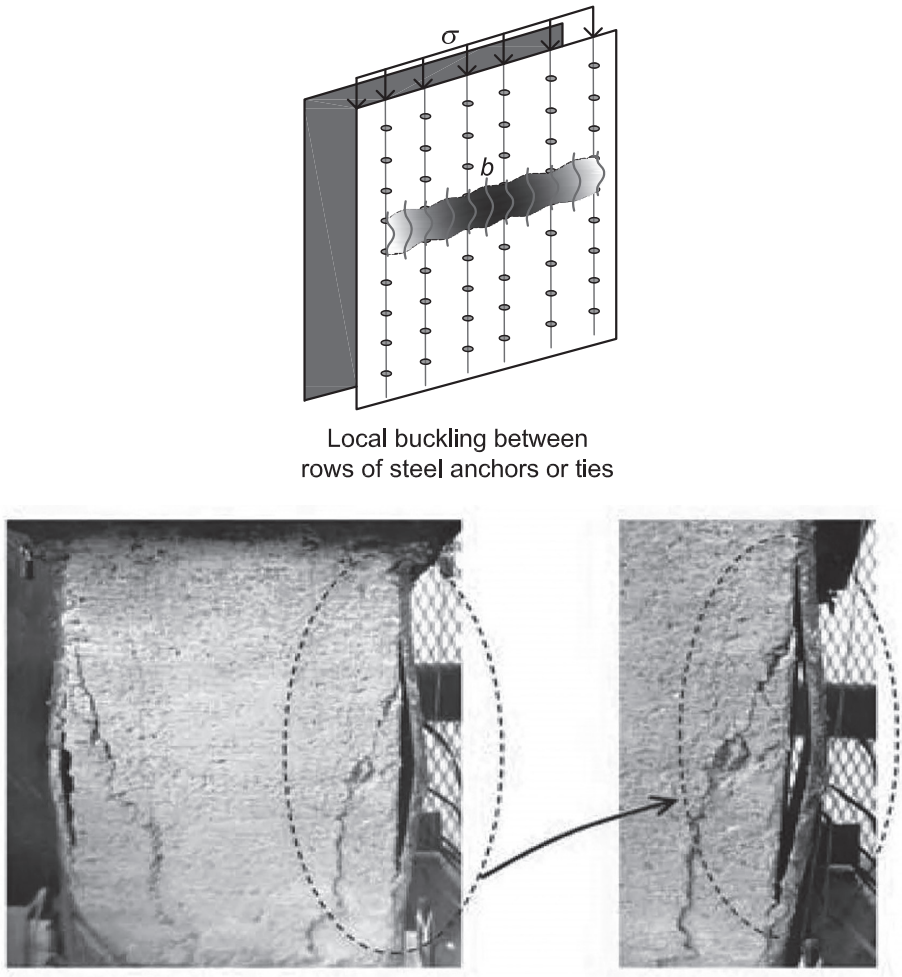


Fig. C-A-N9.1.6. The buckling mode of the faceplate (Zhang et al., 2014).

in nuclear applications, the normalized plate slenderness ratio can be relaxed to 1.20, which is slightly conservative compared to the intersection of the unity $(\varepsilon_{cr}/\varepsilon_y)$ and the Euler's curve (e.g., a K value of 0.7, for which the data fit the Euler's curve reasonably well, yields a coefficient of 1.29). Because ties may also act as steel anchors, Equations A-N9-2a and A-N9-2b consider the largest unsupported length between rows of steel anchors or ties, b.

The faceplate slenderness equations (Equations A-N9-2a and A-N9-2b) 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 (345 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 (345 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 Shear Connectors

The shear connectors used in SC construction may consist of steel headed stud anchors, embedded steel shapes, or ties (e.g., in the form of smooth, deformed, or threaded bars, plates, or shapes that connect the opposite faceplates) that are connected to the faceplates using welded or threaded connections. The slip behavior of shear connectors governs the efficacy of steel-concrete composite action, interfacial shear strength, and slip between faceplates and concrete infill (Zhang et al., 2014).

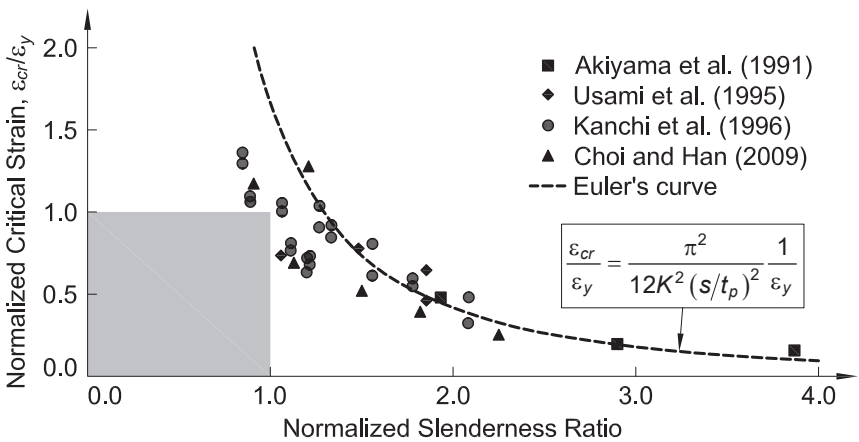


Fig. C-A-N9.1.7. The relationship between buckling strain of plate and normalized slenderness ratio (Zhang et al., 2014).

Shear connectors 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. Shear connectors that have a nonductile shear force-slip behavior cannot redistribute the interfacial shear equally over several connectors. Such connectors 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-N9.1.8). Shear connectors 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 shear connectors 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 other types of shear conenctors unless their behavior is further verified using rigorous benchmarked models and analyses that are based on fewer tests. If necessary, the safety factors applicable for nonyielding shear connectors 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 shear connectors and nonyielding shear connectors 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 shear connectors is limited to the strength corresponding to the interfacial slip at which the nonyielding shear connectors reach their ultimate strength. This is illustrated in Figure C-A-N9.1.9. Based on this construct, the strength of the shear connector system will be the sum of the strengths of individual shear connectors.

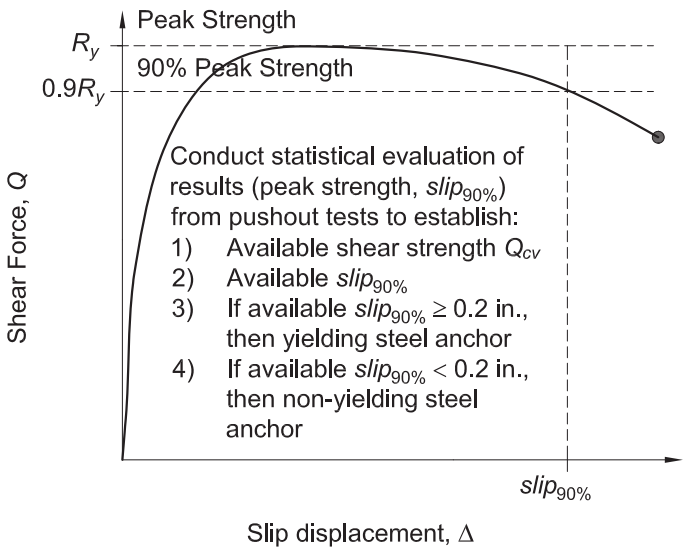


Fig. C-A-N9.1.8. Typical steel anchor force-slip behavior from pushout tests.

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 reinforced concrete structures. The development length, L_d, should be designed to be approximately two to three times the SC structural element's section thickness, t_{sc}, which is the typical development length for No. 11 to No. 18 rebars in reinforced concrete structures. It is often possible and desirable to have a smaller development length in the range of section thickness.

The use of Q_{cv}^{avg}, the weighted average of the available interfacial shear strengths of shear connectors, is appropriate when a combination of various types of shear connectors (e.g., ties and steel headed stud anchors) are present. The Q_{cv}^{avg} concept is illustrated in Commentary N9.3.6a. The Q_{cv}^{avg} term simply devolves to the design shear strength of connector value, Q_{cv}, when only a single type of shear connector is present.

4b. Spacing of Shear Connectors

Figure C-A-N9.1.10 from Zhang et al. (2014) shows the free body diagram that resulted in the spacing requirement for yielding shear connectors to achieve faceplate yielding over the development length, L_d. As shown in Figure C-A-N9.1.11, all the yielding shear connectors in the development length contribute equally to developing the yield strength of the faceplate.

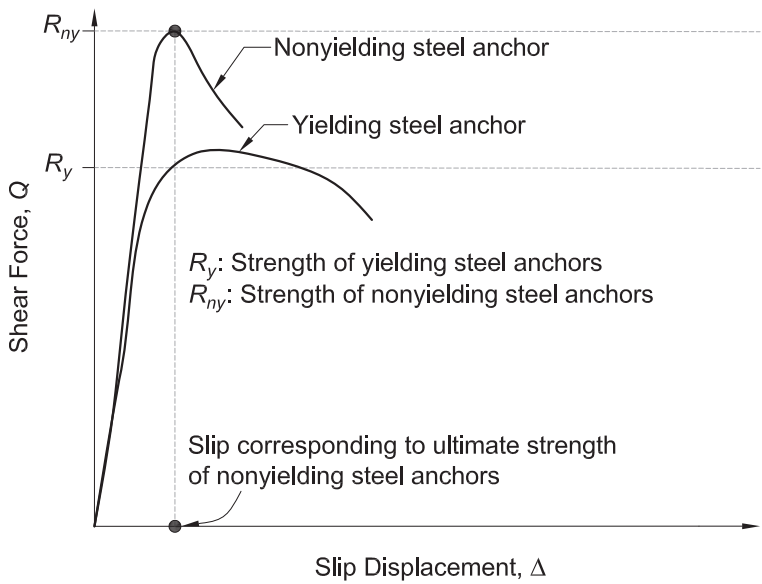
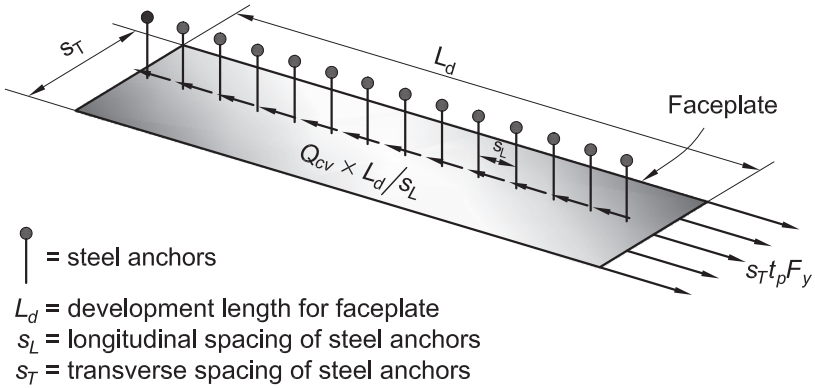


Fig. C-A-N9.1.9. Strength of yielding steel anchors that form a part of nonyielding steel anchor systems.

Both the out-of-plane shear capacity and interfacial shear demand can be quite high for situations when the shear-span ratio is less than 2.5. This is because the concrete provides significantly higher out-of-plane shear contribution in these situations; however, the accompanying interfacial shear demand can also be quite high. Either way, the governing failure mode in these situations, whether controlled by interfacial shear failure or out-of-plane shear failure, is non-ductile, which is something that cannot be avoided. This notwithstanding, for situations when flexure governs (i.e., when the shear-span ratio exceeds 2.5), enhanced flexural ductility is ensured by requiring that the interfacial shear strength of SC structural elements exceeds a



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

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

Fig. C-A-N9.1.10. Spacing requirement for plate yielding for yielding shear connector system.

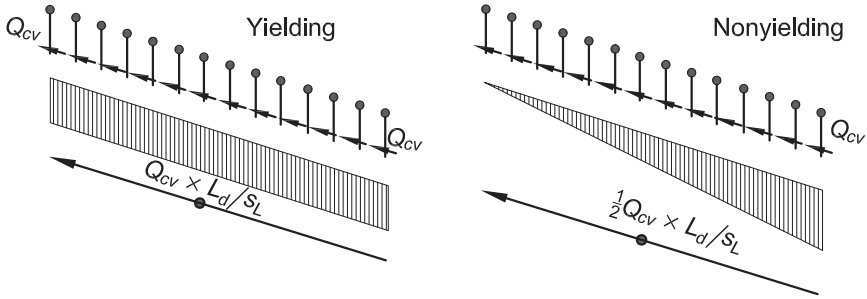
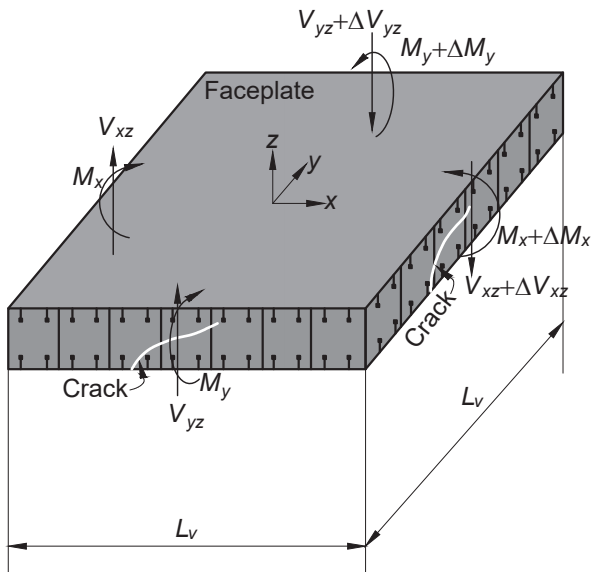


Fig. C-A-N9.1.11. Interfacial shear resistance of yielding and nonyielding shear connector systems (LRFD).

realistic upper bound on the out-of-plane shear strength. This provision prevents interfacial shear failure, which is an SC-specific limit state that is similar to the bond shear failure mechanism in reinforced concrete, from governing the behavior and failure mode. Figure C-A-N9.1.12 shows the free body diagram that resulted in the spacing requirement for yielding shear connectors so that out-of-plane shear failure would occur before interfacial shear failure.

Figure C-A-N9.1.12(a) shows the derivation of the spacing requirement for shear connectors 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.

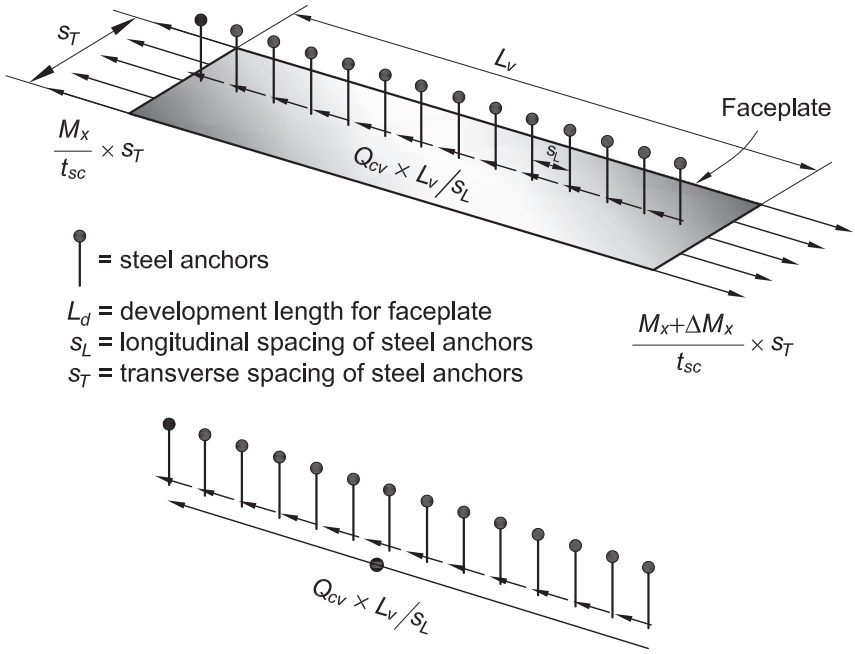
Figure C-A-N9.1.12(b) shows the free body diagram of the bottom faceplate in tension over the length, L_v. The tension forces resulting from the applied moments are included in the figure. The equilibrating force from the yielding shear connectors is calculated as the design shear strength, Q_{cv}, of each connector multiplied by the number of connectors. For situations when flexural behavior controls (i.e., when the shear span-to-depth ratio is greater than 2.5), $M_n/(2.5t_{sc})$ is considered a realistic upper bound on the maximum out-of-plane shear force based on the results



(a) Spacing requirement for shear connectors

Fig. C-A-N9.1.12. Shear connector spacing requirement for preventing interfacial shear failure before out-of-plane shear failure (LRFD).

presented by Sener and Varma (2021). 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-N9.1.12(c).



(b) Free body diagram of the bottom faceplate in tension (in the above figure the term "steel anchors" has been used in lieu of "shear connectors")

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

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

and $\frac{\Delta M_x}{L_v} = V \leq \frac{M_n}{2.5 t_{sc}}$

Substituting these expressions

$$s \leq \sqrt{\frac{Q_{cv}^{avg} t_{sc}}{\frac{M_n}{2.5 t_{sc}}}}$$

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

Fig. C-A-N9.1.12. (cont'd). Shear connector spacing requirement for preventing interfacial shear failure before out-of-plane shear failure (LRFD).

The derivation in Figure C-A-N9.1.12(c) corresponds to the situation when flexural behavior governs; i.e., for the classical interaction theory between out-of-plane shear and flexural moment. Such behavior is applicable when the shear span-to-depth ratio is 2.5 or larger, which is why the lower bound value of 2.5 was considered in the formulation for defining the maximum reasonable out-of-plane shear demand. For these situations, ductile behavior is ensured by requiring that interfacial shear failure will not occur before out-of-plane shear failure, which is accomplished by Equation A-N9-4.

In contrast, for situations when the shear span-to-depth ratio is smaller than 2.5, the potential for a non-ductile interfacial shear failure is not a concern. This is because, unlike the shear-flexure interaction formulation considered for development of Equation A-N9-4, the accompanying flexural and out-of-plane shear interaction behavior for small shear span-to-depth ratios is dominated by arching action and strut-and-tie behavior. Interfacial shear failure is an unlikely (if not irrelevant) limit state during this behavior.

For nonyielding shear conenctors, 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-N9.1.11. This change in the resistance of nonyielding shear connectors in turn impacts their spacing requirements.

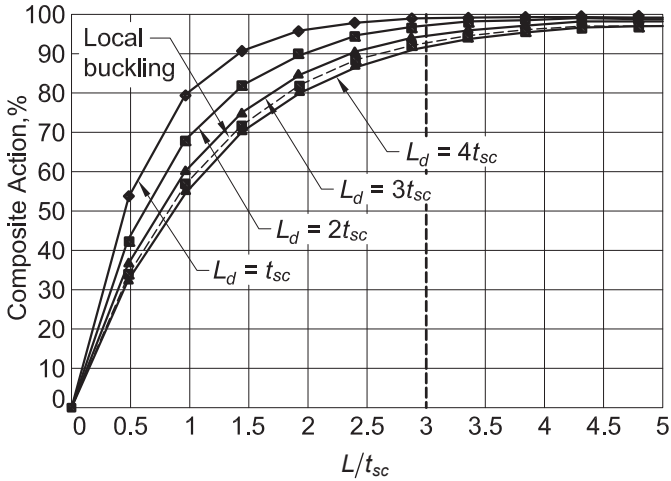
5. Tie Requirements

The ability of the faceplates of SC structural elements 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.

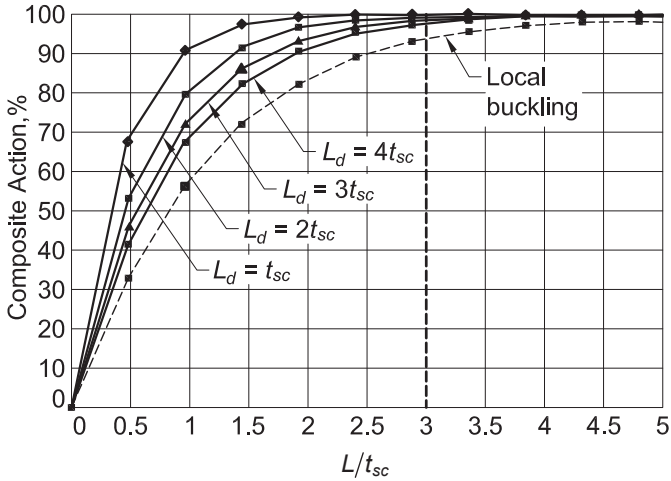
Ties contribute to the out-of-plane shear strength and structural integrity of SC structural elements. Their contribution to the out-of-plane shear strength (according to Section N9.3.5) may be required for the calculated design demands (required strengths). Ties also provide structural integrity in 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 structural elements. Tie spacing can be as large as section thickness, t_{sc}, or 48 times the tie bar diameter [in accordance with ACI 318-19 or ACI 318M-10, Section 25.7.2.1 (ACI, 2019)]. 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 structural elements subjected to axial loading on the concrete only at the ends. As shown in Figures C-A-N9.1.13(a) and C-A-N9.1.13(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 than or equal to at least three times the section thickness, t_{sc}, for SC structural elements with reinforcement ratios of 0.015 to 0.10.

Zhang et al. (2014) show that SC structural elements designed with shear connector 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 shear connectors, 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 shear connectors,



(a) $2t_p/t_{sc} = 0.02$



(b) $2t_p/t_{sc} = 0.04$

Fig. C-A-N9.1.13. Development of strain compatibility with distance from member end (Zhang et al., 2014).

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 structural element designs (faceplates and shear connector size and spacing).

However, the effects of having longer transfer lengths are somewhat inconsequential. The design capacities or available strengths of SC structural elements 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 that the ties have to be designed for.

5a. Classification of Ties

The requirements of this section help ensure that the rupture strength of threaded ties will comfortably exceed their yield strength that is based on the cross-sectional area of the unthreaded portion. These strengths are based on the tensile stress and yield stress, respectively, of the same tie material. It should be noted that Section A3.2 of the *Seismic Provisions* (AISC, 2022b) does not require consideration of R_y and R_t factors for these situations, where R_y is the ratio of the expected yield stress to the minimum specified yield stress, and R_t is the expected tensile strength to the minimum specified tensile strength. Regardless, the Nuclear Specification accounts for the fact that the expected yield stress and tensile stress are always higher than the corresponding nominal values, and that the respective overage may not be identical (i.e., R_y could be larger than R_t). As a result, for a yielding type tie, F_{ny}, the yield strength of the tie, needs to be smaller than (R_t/R_y) times F_{nr}, the tie rupture strength. Note that this consideration is deemed unnecessary for ties welded using full-strength stud welding techniques since this is a mature technology that gives consistent results, and the same material is involved in the stud welding process (i.e., no filler metal is used). For cases involving ties with threaded connections and filler metal welded connections, the following justification is provided for using the 0.85 coefficient as the deemed lower bound of the R_t/R_y values.

It is common to use ASTM A706/A706M (ASTM, 2016) rebar as tie material for threaded (or welded) connections. The R_t/R_y value for A706/A706M is 1.0 (see Table A3.2 in the *Seismic Provisions*). In general, the R_t/R_y value is higher than 0.90; however, a 0.85 factor has been conservatively adopted to account for threaded connections involving high-strength tie materials, such as ASTM A1064/A1064M (ASTM, 2024) grade rebar and high-strength threaded rods. (Note that the R_t/R_y construct does not even apply to all threaded bars since they cannot have a limit state based on yielding of the gross section.)

Similarly, for ties with welded connections using filler metal welds, the 0.85 coefficient represents a conservative R_t/R_y lower limit. This is true for situations that involve limit states for the base metal alone (i.e., when both R_y and R_t values pertain to the base metal), as well as for situations that involve limit states that controlled by either the weld metal (with its R_t value) and base metal (with its R_y value). Stud

TABLE C-A-N9.1-2
Required Tie Spacing

Wall Thickness, in. (mm)	Required Tie Spacing—2018 Edition of the Nuclear Specificaiton, in. (mm)	Required Tie Spacing—2024 Edition of the Nuclear Specification, in. (mm)
24 (600)	16.8 (420)	15.1 (380)
30 (750)	18.7 (470)	13.5 (340)
36 (900)	20.4 (510)	12.3 (310)
48 (1 200)	23.6 (590)	10.6 (270)
60 (1 500)	26.3 (660)	9.5 (240)

welded connections comforming to AWS D1.1/D1.1M (AWS, 2020) are exempt from this requirement because they reliably develop the yield strength of the tie based on its gross area.

5b. Tie Spacing

The tie bar spacing requirement is based on the flexibility and shear buckling of empty steel modules before concrete placement (Varma et al., 2019). Experimental and numerical results indicate that tie bars meeting these requirements provide adequate restraint for local buckling and concrete confinement in the composite phase (Agrawal et al., 2020; Harmon and Varma, 2021; Bhardwaj et al., 2018). Modules that meet the plate slenderness requirements can typically be cast with concrete pour heights of up to 30 ft (9.1 m) without significant influence of induced deflections and stresses on the compressive strength and buckling of the steel plates.

Table C-A-N9.1-2 shows a comparison of the required tie spacing using the strength requirement in the previous edition of the Nuclear Specification and the spacing requirement in the current edition. It shows that the spacing requirement is now more stringent.

6. Design and Detailing Requirements for Impactive and Impulsive Loads

Refer to the Commentary for Appendix N10.

7. Design and Detailing Requirements for 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). Note that holes used for rebar dowels or other jointing instruments do not constitute as openings.

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.

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. Additionally, the literature shows that very small openings, those with largest dimension equal to or less than 6 in. (150 mm), but not exceeding $t_{sc}/4$, have little effect in terms of an increase in the localized membrane/bending demands and the associated localized capacity reduction, as long as such very small openings are not closely spaced.

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

7a. Design and Detailing Requirements for Small Openings

As noted in the foregoing discussion, very small openings have little adverse impact on the membrane/bending force demands and the associated capacity reductions. Accordingly, very small openings (smaller than 6 in. diameter, but not exceelding $t_{sc}/4$) require modest detailing in the form of rounded corners and incorporation of an anchored penetration sleeve, to ensure continuity of the faceplates at the opening perimeter.

To help ensure 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) Analysis, design, and detailing for free edge at opening perimeter

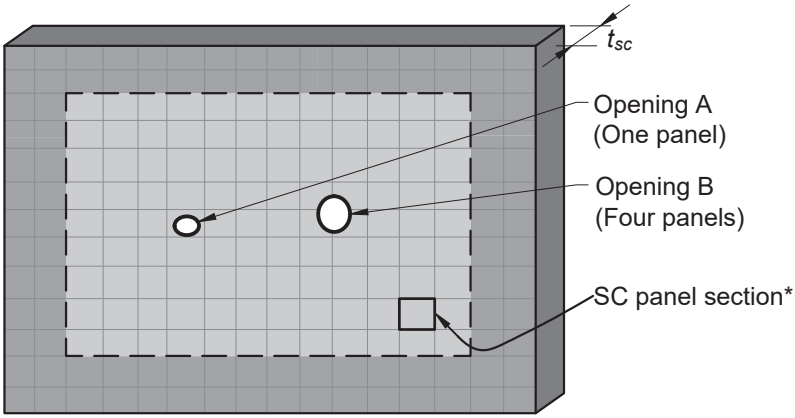
Experiments conducted by Japanese researchers (Ozaki et al., 2004) indicate that the maximum decrease in SC panel section capacity is about 15 to 20%.

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-N9.1.14), 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-N9.1.14), the strength of all panel sections that partially include the opening will be reduced by 25%.

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-N9.1.15 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) Analysis, 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 shear 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.



*Depending on the degree of mesh refinement, the notional SC panel section may or may not be the same as the element in the finite element mesh.

Fig. C-A-N9.1.14. Reduction in strength due to the presence of an opening.

The detailing for the sleeve can be thought of as a cylinder spanning across the SC structural element's cross-section with annular discs at its two edges. The flange plate is extended a minimum distance of one times the SC structural element's 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:

- In the case that a doubler plate is used, the doubler plate helps carry the concentrated stresses around the opening. The doubler plate is mounted outboard of the native faceplate and with the same thickness as the faceplate. Also, the doubler plate is connected with the sleeve using a complete-joint-penetration (CJP) groove weld around its inner perimeter, and at its outer perimeter, it is connected with the native faceplate using the maximum size fillet weld permitted by the *Specification*. (See Figure C-A-N9.1.16.)
- In the case that an independent reinforcing plate is used, the reinforcing plate thickness needs to be at least 1.25 times the faceplate thickness, and it is welded to the sleeve using a CJP groove weld. In this option, the native faceplates mate only with the reinforcing flange at its outer perimeter. Therefore, the entire penetration assembly (i.e., comprising of the sleeve, reinforcing plate, and closely spaced ties around the sleeve) can be fabricated separately, then connected with the surrounding native faceplates. (See Figure C-A-N9.1.17.)

No reduction in SC panel section capacities is considered because of exercising either of these 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.

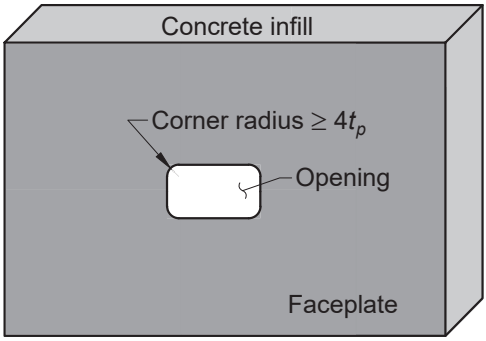


Fig. C-A-N9.1.15. Radius of reentrant corners (elevation view of the SC panel section).

7b. Design and Detailing Requirements for 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 structural element 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-N9.1.18). For example, under this free edge option, a 4-ft- (1.2-m-) diameter circular opening in a 4-ft- (1.2-m-) thick SC structural element will have to be modeled as a 12-ft- (3.7-m-) diameter 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).

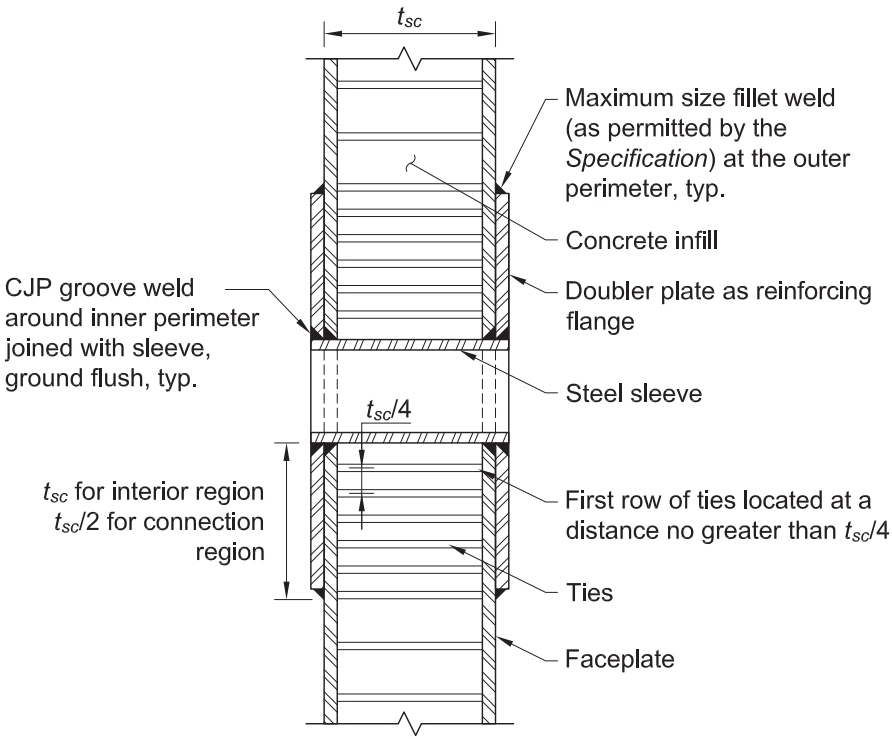


Fig. C-A-N9.1.16. Illustration of detailing around openings using a doubler plate as reinforcing flange.

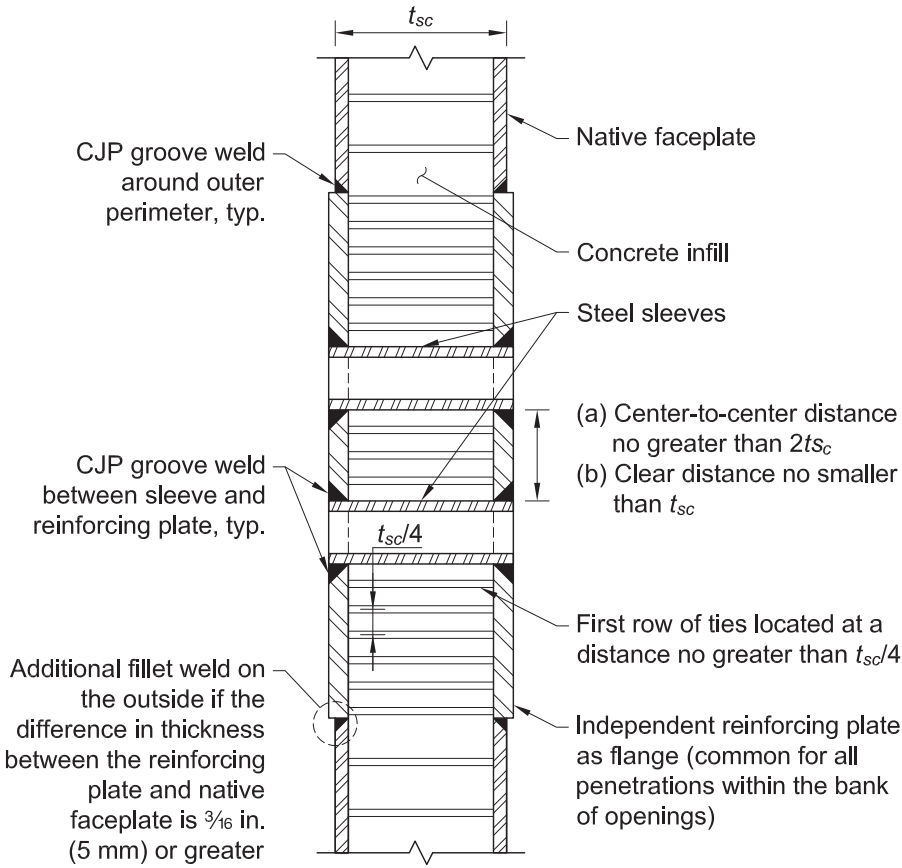


Fig. C-A-N9.1.17. Illustration of detailing around openings using an independent reinforcing plate as fange.

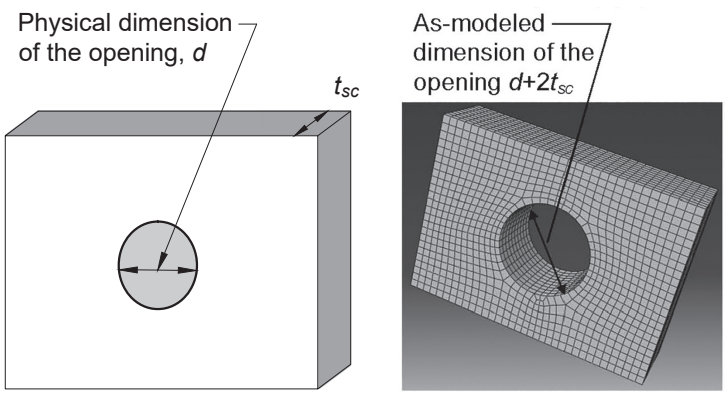


Fig. C-A-N9.1.18. Modeling of large openings with free edge at opening perimeter.

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 structural element. 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.

(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.

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

The detailing requirements for a bank of very small openings [less than 6 in. (150 mm) in diameter, but not exceeding $t_{sc}/4$] are not onerous provided that their center-to-center spacing is at least t_{sc}. This is because the associated group effect, in terms of impact on increased localized demands and reduced strength, is considered to be small, provided that the modest detailing requirements for very small openings are implemented. The bank of very small openings is detailed in the same manner as a bank of small openings when the center-to-center spacing between adjacent very small openings is less than t_{sc}.

Special detailing requirements are triggered for a bank of small openings if the smallest clear distance between adjacent openings is between t_{sc} and $2t_{sc}$ for the interior region and $0.5t_{sc}$ and $1.5t_{sc}$ for the connection region. The special detailing consists of a common reinforcing plate per Section N9.1.7a(b)(4)(ii) that caters to all of the openings and all penetration sleeves that are connected to the reinforcing plate. (See Figure C-A-N9.1.17.)

Analysis/modeling needs for a bank of small openings depend on the overall expanse of the bank relative to SC structural element's thickness. This helps address egregious situations when the overall bank dimensions become rather large. Accordingly, a bank of small openings is required to be analyzed per Section N9.1.7b as an equivalent single "large opening" that circumscribes the outermost sleeves if both the longest and shortest dimensions of the bank exceed $2t_{sc}$ and t_{sc}, respectively (see Figure C-A-N9.1.19).

N9.2. ANALYSIS REQUIREMENTS

1. General Provisions

SC structures are modeled using elastic finite elements, as explained earlier in Commentary 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 (SSE) 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 (OBE) 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 structures, 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/SEI 4-16 (ASCE, 2016) 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 structural elements

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 structural elements. 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.

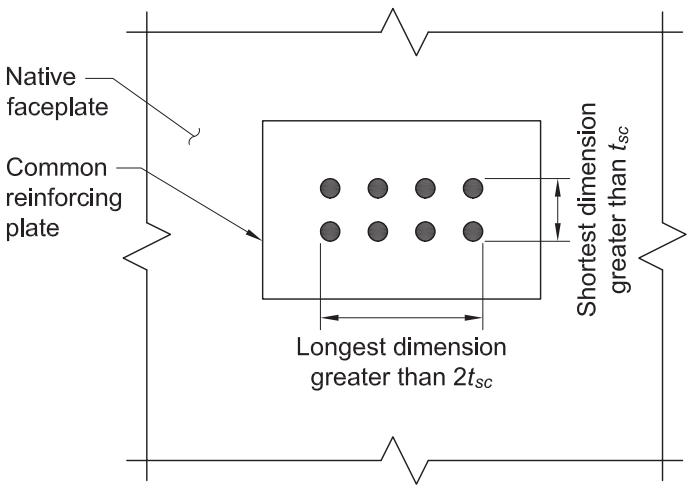


Fig. C-A-N9.1.19. Illustration of condition when analysis is required for a bank of small openings.

The cracked-transformed flexural stiffness of the SC section, $(EI)_{cr\text{-}tr}$, 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-N9-4a).

Equilibrium of forces in Figure C-A-N9.2.1 results in Equation C-A-N9-2a for neutral axis depth, wherein ρ' is the stiffness-normalized reinforcement ratio (Equation C-A-N9-3).

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

$$\rho' = \frac{2t_p E_s}{t_{sc} E_c} \qquad \text{(C-A-N9-3)}$$

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

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

However, Varma et al. (2011a) calibrated this equation to the simpler form given by Equation C-A-N9-5:

$$(EI)_{cr\text{-}tr} = E_s I_s + c_2 E_c I_c \qquad \text{(C-A-N9-5)}$$

where

$$c_2 = 0.48\rho' + 0.10 \qquad \text{(C-A-N9-6)}$$

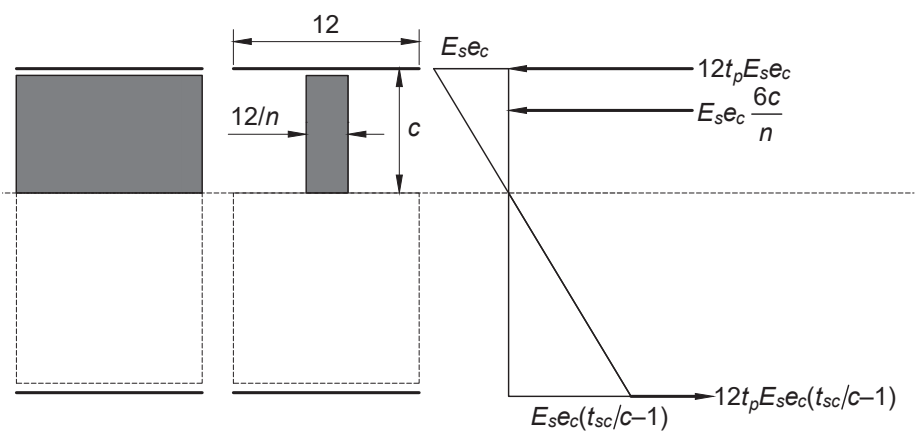


Fig. C-A-N9.2.1. Flexural stiffness of cracked-transformed section of SC elements.

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-N9-2b and C-A-N9-4b. It is observed that the values using these equations match closely with those obtained using the simplified method (Equations C-A-N9-2a and C-A-N9-4a).

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

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

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'.

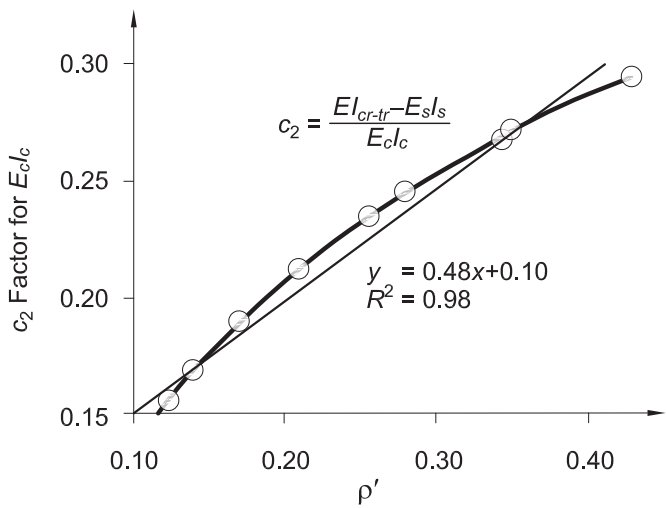


Fig. C-A-N9.2.2. Calibration of c_2 *versus* ρ' *(Varma et al., 2011a).*

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 $\left(\Delta T_{s1}^{max} \text{ and } \Delta T_{s2}^{max}\right)$ due to accident thermal conditions.

$$\Delta T_{savg} = \frac{\left(\Delta T_{s1}^{max} + \Delta T_{s2}^{max}\right)}{2} \tag{C-A-N9-7}$$

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

The in-plane shear behavior of SC structural elements 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 sections with reinforcement ratios, ρ, from 0.015 to 0.050. This model is discussed in Commentary N9.3.4.

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

$$S_{cr} = \left(\frac{0.126\sqrt{f_c'}}{G_c} - \varepsilon_{sh}\right)\left(G_cA_c + GA_s\right) \tag{C-A-N9-8}$$

$$S_{cr} = \left(\frac{0.34\sqrt{f_c'}}{G_c} - \varepsilon_{sh}\right)\left(G_cA_c + GA_s\right) \tag{C-A-N9-8M}$$

where

A_c = area of concrete infill, in.^{2} (mm^2)
A_s = gross area of faceplates, in.^{2} (mm^2)
G_c = shear modulus of concrete, ksi (MPa)
ε_{sh} = shrinkage strain

Figure C-A-N9.2.3 shows a plot of experimental versus calculated values of cracking strength by Varma et al. (2014). The concrete cracking shear strength, S_{cr}, is calculated assuming the shrinkage strain, ε_{sh}, to be $0.063\sqrt{f_c'}/G_c$ $\left(\text{SI: } 0.17\sqrt{f_c'}/G_c\right)$. 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

$$K_s = G2t_p \tag{C-A-N9-9}$$

$$K_{sc} = \frac{1}{\dfrac{4}{0.7E_c t_c} + \dfrac{2(1-\nu)}{E_s 2t_p}} \tag{C-A-N9-10}$$

where

K_s = contribution of faceplates to in-plane shear stiffness, kip/in. (N/mm)

K_{sc} = contribution of cracked orthotropic concrete to in-plane shear stiffness, kip/in. (N/mm)

ν = Poisson's ratio of steel

However, under seismic loading, the cyclic behavior of SC sections 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.

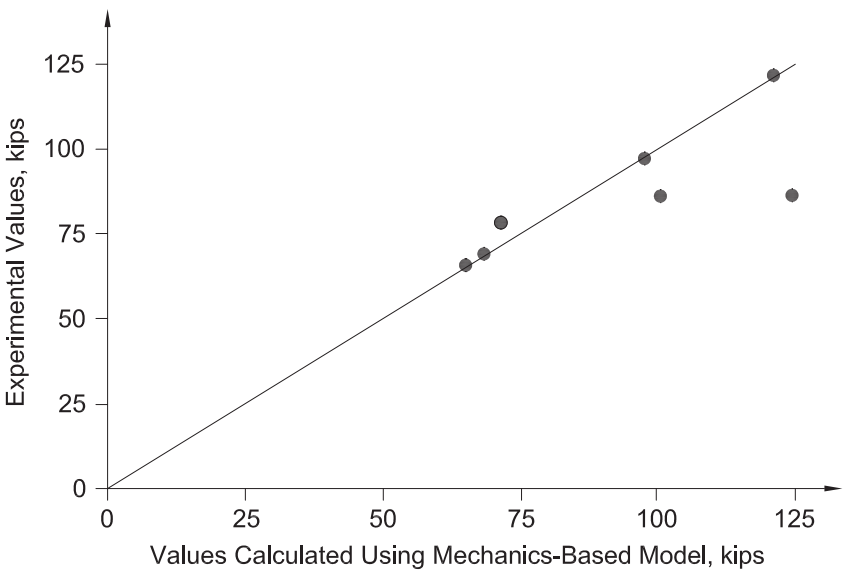


Fig. C-A-N9.2.3. Experimental versus calculated values of concrete cracking shear strength S_{cr} *(Varma et al., 2014).*

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

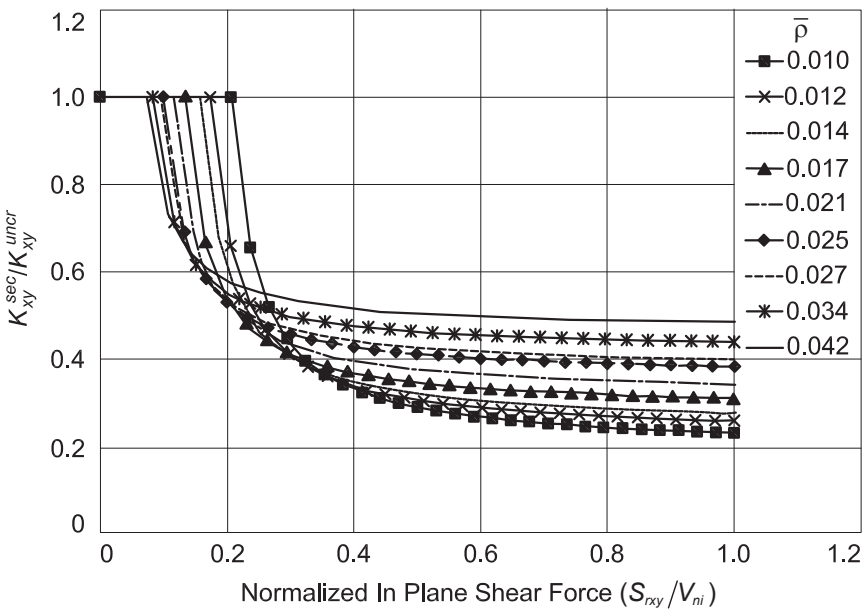


Fig. C-A-N9.2.4. Variation of secant stiffness of SC structural elements (Varma et al., 2011a).

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

The in-plane shear stiffness of SC structural elements 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 structural elements after accident thermal loading can be estimated as the post-cracking shear stiffness of the composite section, $K_s + K_c$, i.e.,

$$K_{xy}^{cr} = 0.5\left(\bar{\rho}^{\,-0.42}\right)GA_s \qquad \text{(C-A-N9-11)}$$

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

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.

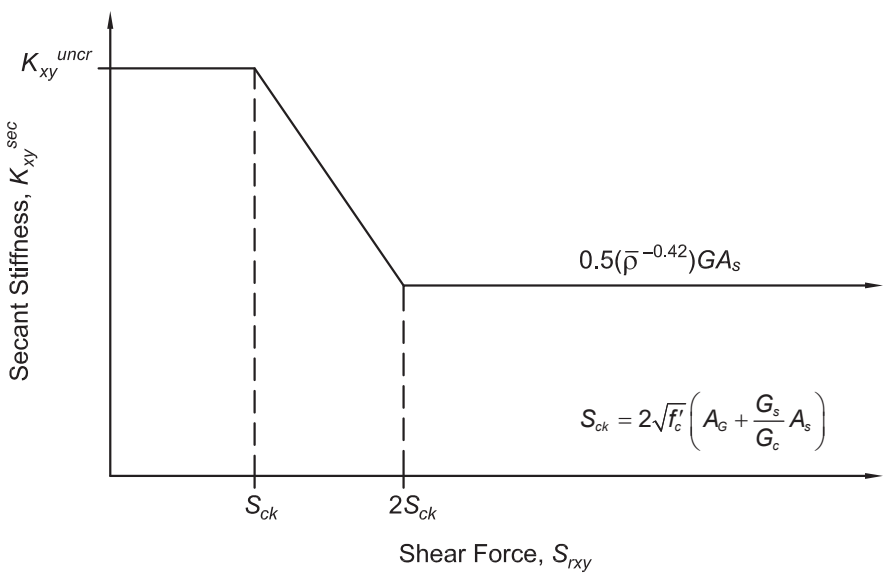


Fig. C-A-N9.2.5. A simple model for secant stiffness with no accident thermal loading (Varma et al., 2011a).

(b) Calibrate the model section thickness and material elastic modulus so that the effective stiffness of the model match those of the physical SC 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 Normal Operating and Accident Thermal Conditions

The temperature limits are based on existing test data for steel and concrete material behavior and SC structural element behavior at accident temperatures (NRC, 2006; Kitajima et al., 2017; Bhardwaj et al., 2023; Bhardwaj et al., 2019). Adequate test data does not currently exist to establish higher temperature limits. To evaluate conditions with higher temperature limits, testing may be required to ensure acceptable structural behavior. Testing should simulate the actual, expected, operating conditions for the intended use. Note that temperature limits given in this section relate to the steel plate surface and not the air temperature, pipe surface temperature, or the temperature at some internal point.

Inelastic strain up to $2\varepsilon_y$ permitted under accident thermal gradient for non-pressure applications is considered acceptable based on ASME Section III Division 2 provisions (ASME, 2023).

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 structural elements. 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 structural elements 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.

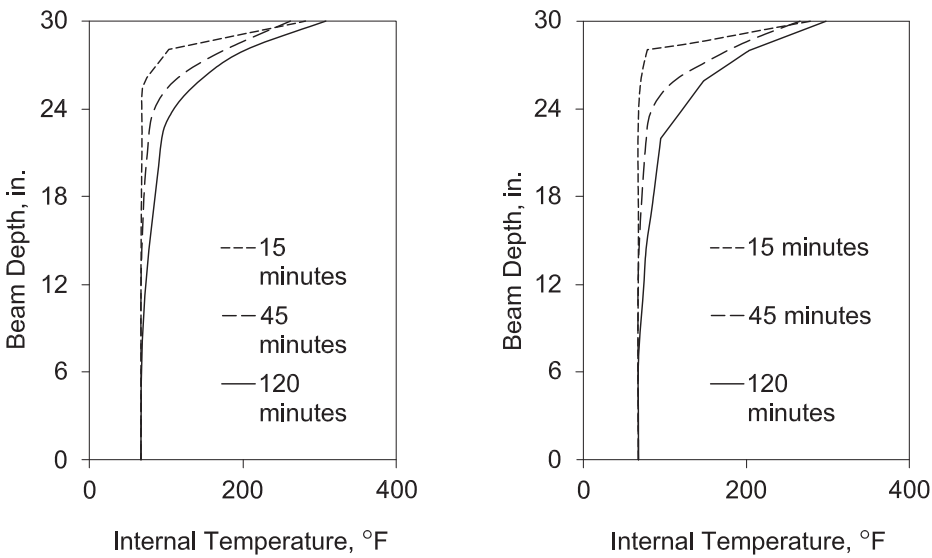
The stiffness of the SC structural element subjected to ΔT_{savg} greater than or equal to 150°F (83°C) can be predicted using fully cracked (steel only) section properties. Based on the preceding results, Varma et al. (2009) developed the simple equations given in the Nuclear Specification to predict 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.

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 section thicknesses, e.g., for a 4-ft- (1.2-m-) thick SC structural element, 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 structural elements. 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 sections 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.



(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).

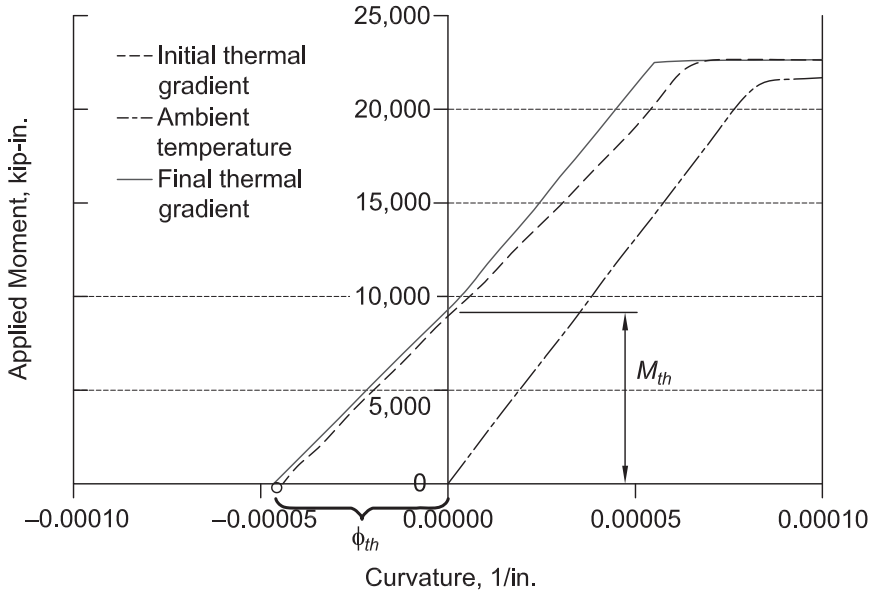


Fig. C-A-N9.2.7. Comparison of fiber model moment curvature to transformed cracked and fully cracked moment of inertia (Varma et al., 2009).

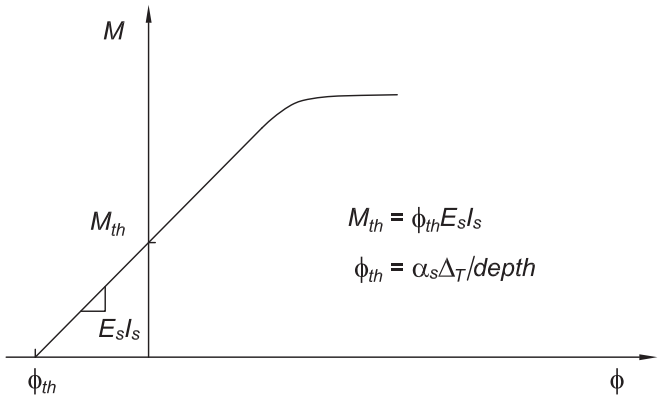


Fig. C-A-N9.2.8. Relationship between moment and thermal gradient.

N9.3. DESIGN OF SC STRUCTURAL ELEMENTS

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.

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 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 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 structural element.

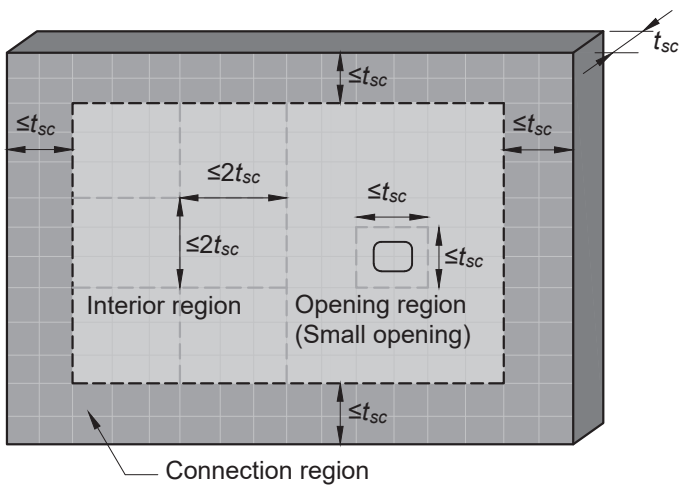
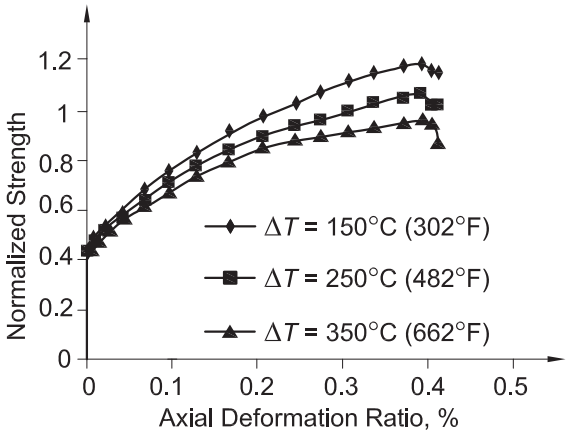
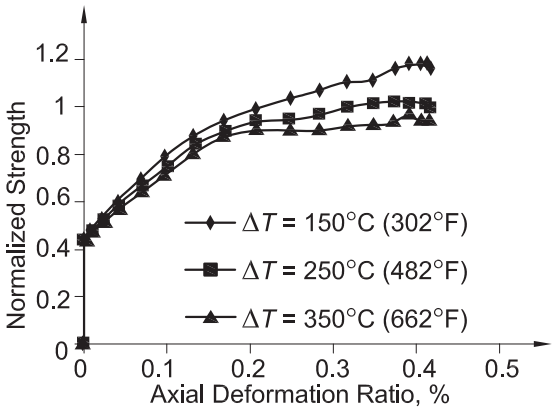


Fig. C-A-N9.2.9. Panel section sizing for averaging the design demands.

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 sections. Therefore, Equation A-N9-16 is recommended for calculating the available compressive strength of SC structural element panel sections subjected to accident thermal loading causing surface temperatures up to 300°F (150°C).



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

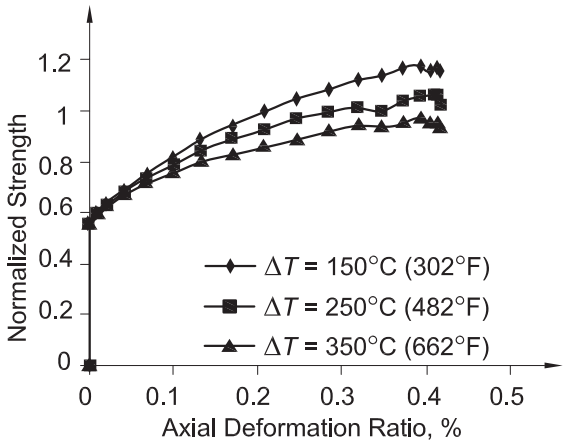


(b) $s/t_p = 20$, *time = 30 minutes*

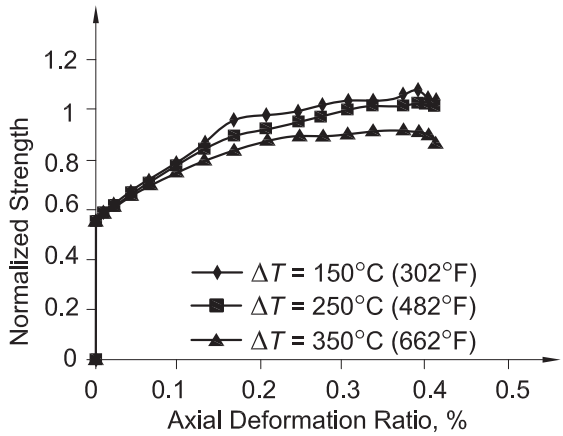
Fig. C-A-N9.3.1. Load displacement curves: temperature magnitude as parameter (Varma et al., 2013).

3. Out-of-Plane Flexural Strength

The nominal flexural strength, M_n, is calculated by assuming that (i) the tension faceplate is yielding, and (ii) the lever arm between the centroid of the tension faceplate and the compression force resultant is equal to 0.9 times the section depth. The effective yield stress of the tension faceplate is also increased by 10% to account for the Poisson effect occurring during uni-axial tension. The final form of the flexural strength equation conservatively predicts the flexural strength of SC beam specimens with a wide range of geometric and material parameters (Sener et al., 2015). 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).



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



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

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

The design assumptions and limitations for determining flexural capacity of concrete members listed in the section can be applied to SC sections 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).

Ignoring the contribution of steel ribs, Equation C-A-N9-12 gives the resultant expression, where c_c is the depth of the triangular concrete compressive stress block.

$$M_n = \left[A_s^F F_y \left(t_{sc} - t_p \right) - \frac{1}{2} f l c_c \left(\frac{c_c}{3} + \frac{t_p}{2} \right) \right] \qquad \text{(C-A-N9-12)}$$

where

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

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

$$c_c = 2t_p \left(\frac{F_y}{f_c'} - n \right) \geq 0 \qquad \text{(C-A-N9-13)}$$

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

l = 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)

Sener et al. (2015) compared the nominal flexural strength values obtained using Equation C-A-N9-12 (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-N9-12. 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 SC structural elements is governed by the plane stress behavior of the plates and the orthotropic elastic behavior of concrete cracked in principal tension. Varma et al. (2014) and Seo et al. (2016) discuss the in-plane shear behavior of composite plate shear walls. The in-plane shear strength of SC

sections can be estimated as the trilinear shear force-strain curve shown in Figure C-A-N9.3.3. The shear force corresponding to the onset of yielding of the steel plates is obtained by Equation A-N9-20. The corresponding principal compressive stress in the cracked (orthotropic) concrete is less than $0.7f_c'$ for typical composite walls with reinforcement ratios $(2t_p/t_{sc})$ less than or equal to 10%. For walls with very high reinforcement ratios (i.e., walls with very thick steel plates compared to overall thickness), the concrete principal compressive stress can be the limiting failure criterion (Seo et al., 2016; Varma et al., 2014). The ultimate shear strength of composite walls, beyond the yielding of the steel plates, depends on the compression strut failure of the cracked concrete infill and can be substantially (20 to 30%) higher (Booth et al., 2020).

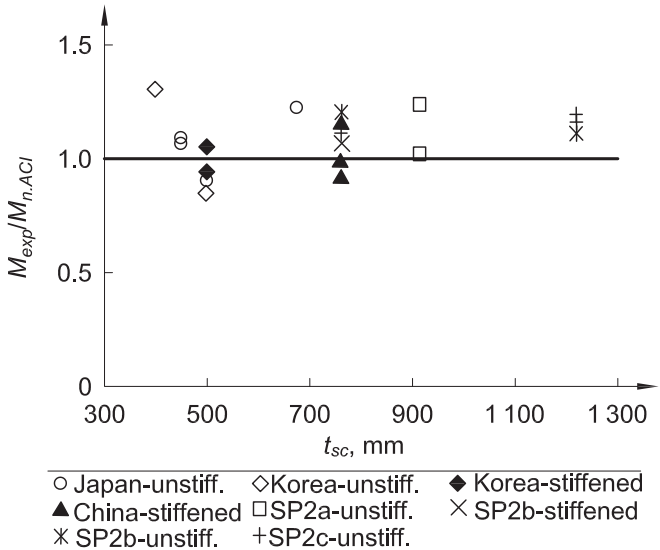


Fig. C-A-N9.3.2. Comparison of experimental flexural strength data with strength using modified Equation C-A-N9-12 (Sener et al., 2015).

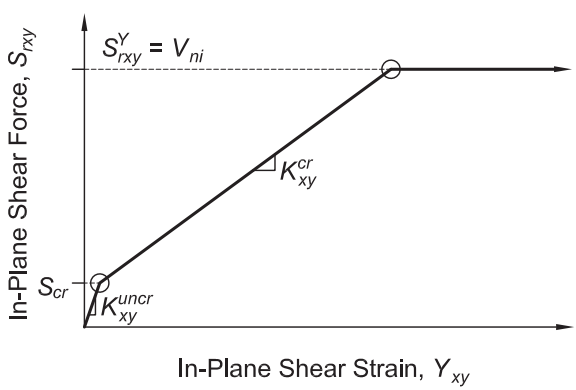


Fig. C-A-N9.3.3. In-plane shear strain curve (Varma et al., 2011b).

For beyond-design basis evaluations, the in-plane shear strength of the SC sections can be increased. Booth et al. (2020) developed an equation for estimating the ultimate in-plane shear strength of SC structural elements. The equation is based on the in-plane behavior mechanics-based model for SC structural elements developed by Ozaki et al. (2004) and Varma et al. (2011b). The equation also includes an additional in-plane shear strength of SC structural elements contributed by the diagonal compressive strength of the concrete infill after cracking. The ultimate in-plane shear strength of SC structural elements can be estimated as the modified trilinear shear force-strain curve shown in Figure C-A-N9.3.4. The first and second parts of the curve are identical to those in Figure C-A-N9.3.3. However, the third part in Figure C-A-N9.3.4 corresponds to the additional in-plane shear strength of SC structural elements contributed by the diagonal compressive strength of the concrete infill. Note, the additional in-plane shear strength cannot be used for any evaluation at elevated temperatures (Bhardwaj et al., 2023).

The ultimate in-plane shear strength, V_{nu}, of the section is given by

$$V_{nu} = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}}\left(2t_p F_y\right) + 0.5\left(0.5f_c' - f_{cy}\right)t_c \qquad \text{(C-A-N9-14)}$$

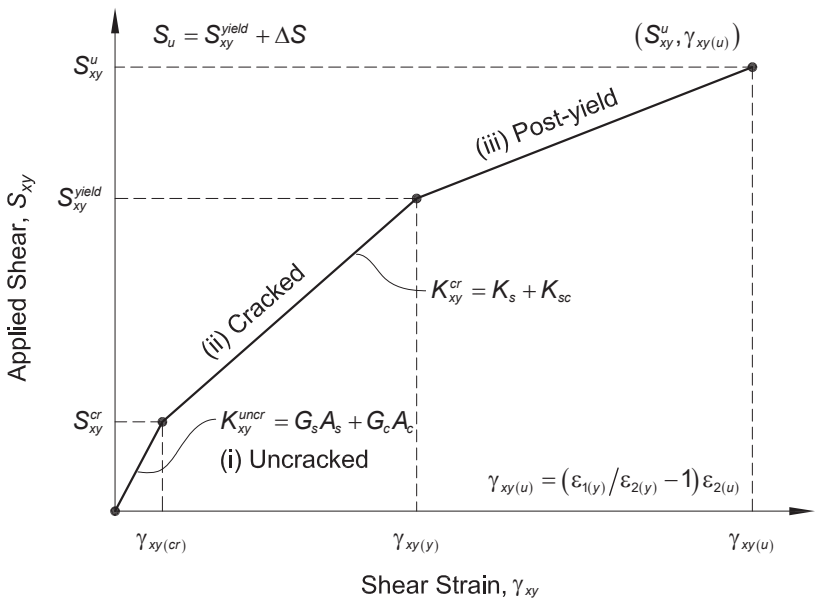


Fig. C-A-N9.3.4. Modified in-plane shear strain curve (Booth et al., 2020).

where

$$f_{cy} = \left(\frac{-S_{xy}^{cr}\left(\nu_c+1\right)\left(\nu_s+1\right)}{2t_p E_s\left(\nu_c+1\right)+E_c t_{sc}\left(\nu_s+1\right)} - \frac{\left(\nu_s+1\right)\left(S_{xy}^{y}-S_{xy}^{cr}\right)}{2t_p E_s + t_{sc}E_c'} \right) E_c' \qquad \text{(C-A-N9-15)}$$

$$S_{xy}^{cr} = \frac{0.063\sqrt{f_c'}}{G_c}\left(G_s 2t_p + G_c t_c\right) \qquad \text{(C-A-N9-16)}$$

$$S_{xy}^{cr} = \frac{0.17\sqrt{f_c'}}{G_c}\left(G_s 2t_p + G_c t_c\right) \qquad \text{(C-A-N9-16M)}$$

$S_{xy}^{y} = V_{ni}$

E_c = secant elastic modulus of the concrete infill, ksi (MPa)

E_c' = effective concrete compression stiffness, ksi (MPa)
= $0.7E_c$

E_s = elastic modulus of the steel faceplates, ksi (MPa)

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

t_c = concrete infill thickness, in. (mm)

t_p = thickness of faceplate, in. (mm)

ν_c = Poisson's ratio of the concrete infill

ν_s = Poisson's ratio of the steel faceplates

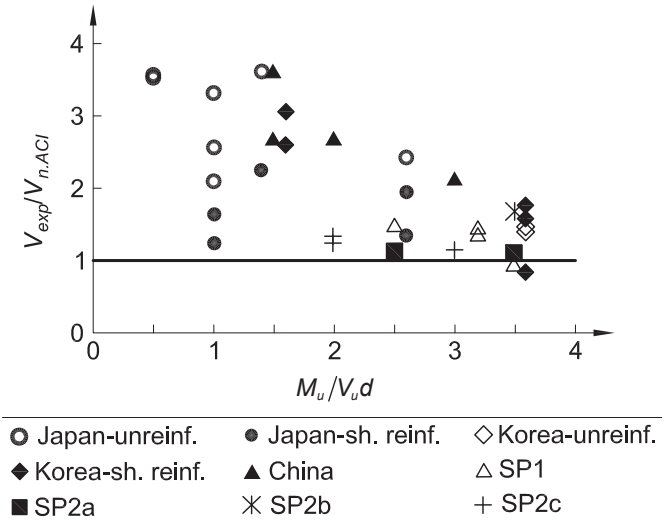
5. Out-of-Plane Shear Strength

The out-of-plane shear behavior of SC structural elements 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 sections. 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, 2021) have compared the shear strengths obtained from this experimental database with the ACI 349-13 or ACI 349M-13 (ACI, 2013) shear strength equations.

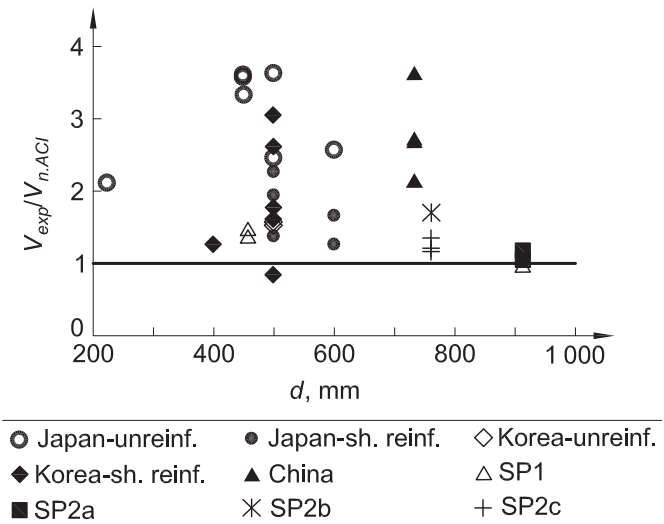
Figure C-A-N9.3.5(a) shows the plot of shear strengths obtained from the specimen, normalized with the strength from ACI provisions, and varying with shear span-to-depth ratios. There is a clear trend in the plot where the increase in shear span-to-depth ratio results in a decrease in the strength of 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-N9.3.5(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 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). No upper limit is given for shear strength contribu-

tion from shear reinforcement (V_s) because test data (Sener and Varma, 2014) do not show such limit is applicable to SC structural elements. In contrast, Section N9.3.5(b), which corresponds to the situation when the tie spacing is greater than half the section thickness, requires the nominal shear to be based on greater of the concrete contribution and steel contribution (i.e., not their sum).



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



(b) Variation with section depth

Fig. C-A-N9.3.5. Comparison of experimental out-of-plane shear strength data with strength using ACI equations (Sener and Varma, 2014).

Similar to the case for filled composite members in the *Specification*, the resistance factor for SC structural elements is specified as 0.90 for sections with yielding ties as long as the section's nominal shear strength is not controlled by the concrete contribution [which can happen for the case addressed in Section N9.3.5(b), i.e., when the tie spacing is greater than half the section thickness], in which case the resistance factor is reduced to 0.75. This reduction in the value of resistance factor represents the higher degree of uncertainty and scatter in the test data when the concrete contribution controls the section's out-of-plane shear strength. However, in SC design, the V_s contribution from ties is typically significantly larger than V_{conc}, the concrete contribution. Accordingly, the default resistance factor for out-of-plane strength is defined as 0.90 provided that the ties are classified as yielding [except for when the case in Section N9.3.5(b) applies and the concrete contribution to out-of-plane shear strength exceeds the steel contribution]. The 0.90 resistance factor value is justified because the contribution of yielding ties has low uncertainty (as is typical for all steel-governed ductile failure modes—e.g., the resistance factor for flexural strength is also specified as 0.90).

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 is taken as $0.063\sqrt{f_c'}$ in ksi $\left(0.166\sqrt{f_c'}\text{ in MPa}\right)$, which is equal to the shear strength provided in ACI provisions.

When the spacing of the yielding shear reinforcement is greater than half the section thickness, the nominal 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. Interaction Criteria for SC Structural Elements Subjected to Concurrent In-Plane and Out-of-Plane Forces

Individual load cases, as well as the specified load combinations, produce load effects that can include concurrent in-plane shear force, bi-axial membrane forces, out-of-plane shear forces, out-of-plane moments, and torsion. Section N9.2 provides the analysis requirements, including provisions for adjustment of effective stiffnesses under operating thermal and accident thermal conditions. In particular, accident thermal loading results in significant cracking, which in turn leads to reduced element stiffness. [See Sections N9.2.2(a) and N9.2.2(c) for determining the reduced bending and in-plane shear stiffness values, respectively.] This in turn produces smaller element force demands because the demands associated with thermal loading are directly related to element stiffness (and the degree of internal or external constraint against thermally induced strains). It is possible to perform sophisticated analyses to determine the demands due to accident thermal loading; however, such effort, which requires significant expertise, may not be worthwhile because the simpler analysis requirements per Section N9.2 adequately capture the reduction in demand due to reduced effective stiffnesses (Bhardwaj, 2018).

Sections N9.3.1 to N9.3.5 define the available strengths for various types of load effects. It is noted that, unlike its influence on the effective stiffness, postulated design-basis accident thermal conditions for safety-related nuclear facilities have little influence on the various types of strengths of SC structural elements (Bhardwaj et al., 2019; Bhardwaj et al., 2023). Accordingly, the interaction equations presented in Section N9.3.6 can be utilized for load combinations involving both operating thermal and accident thermal load cases.

6a. Interfacial Shear and 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, an interaction, with the exponent of 5/3 for nonyielding shear reinforcement and 2 for yielding shear reinforcement, is assumed and the shear reinforcement is checked to ensure that it is not overstressed (yielded) by the combinations of demands. The exponent of 2 is established using von Mises criterion for the yielding reinforcement.

In the first part of the 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:

$$Q_{cv}^{avg} = \frac{n_{et}Q_{cv}^{tie} + n_{es}Q_{cv}}{n_{et} + n_{es}} \qquad \text{(C-A-N9-17)}$$

where

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

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

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

The unit cell is the quadrilateral region between four ties. It is illustrated in Figure C-A-N9.3.6 for an SC structural element 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 [(1)(9) + (0.5)(12)] = 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.

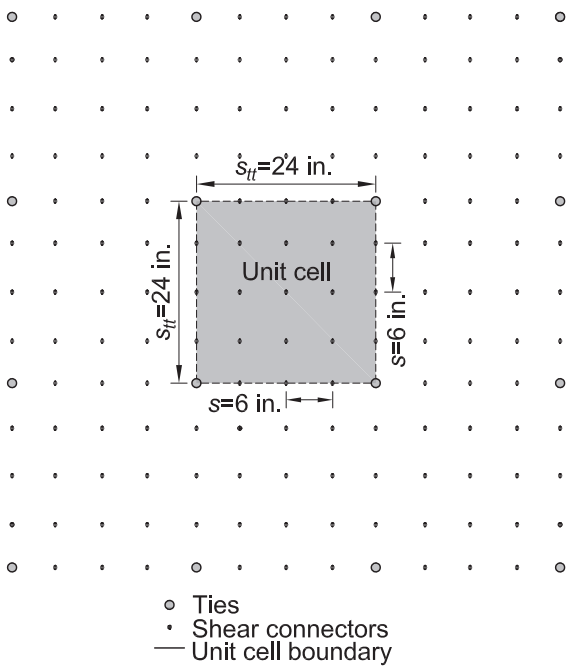


Fig. C-A-N9.3.6. Unit cell for calculating Q_{cv}^{avg}.

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-N9.3.7 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, (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-N9.3.7. 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 structural element panel section. As shown in Figure C-A-N9.3.8, 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).

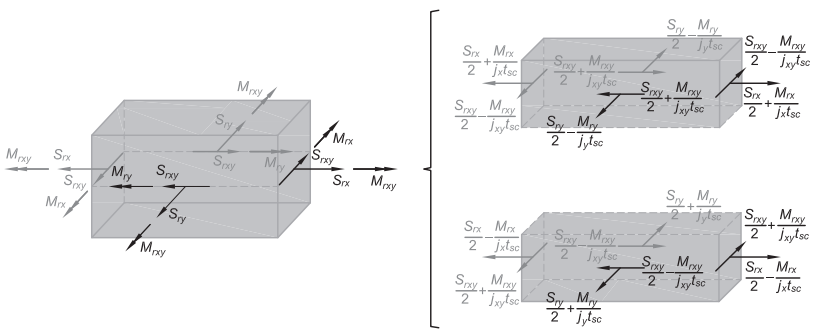


Fig. C-A-N9.3.7. Combined forces acting on panel section and notional halves (Varma et al., 2014).

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.

As shown in Figure C-A-N9.3.9, the uniaxial tensile strength is conservatively adjusted to be collinear with the straight line joining the anchor points of pure

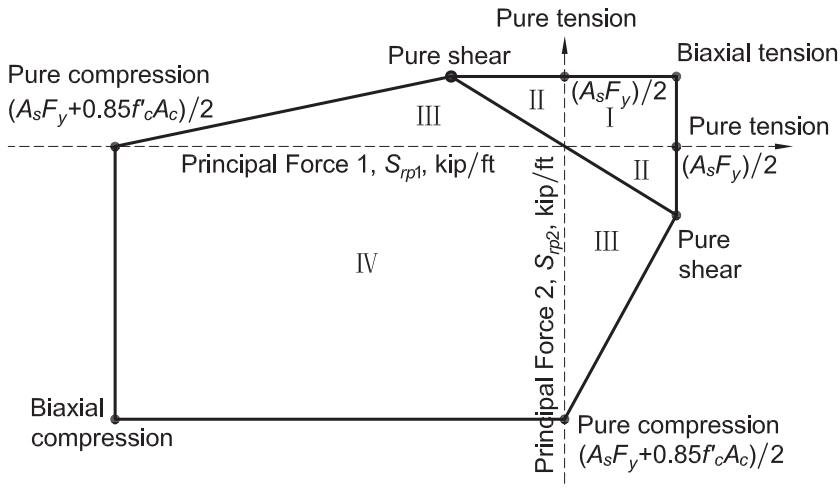


Fig. C-A-N9.3.8. Interaction surface for in-plane forces in principal force space (Varma et al., 2014).

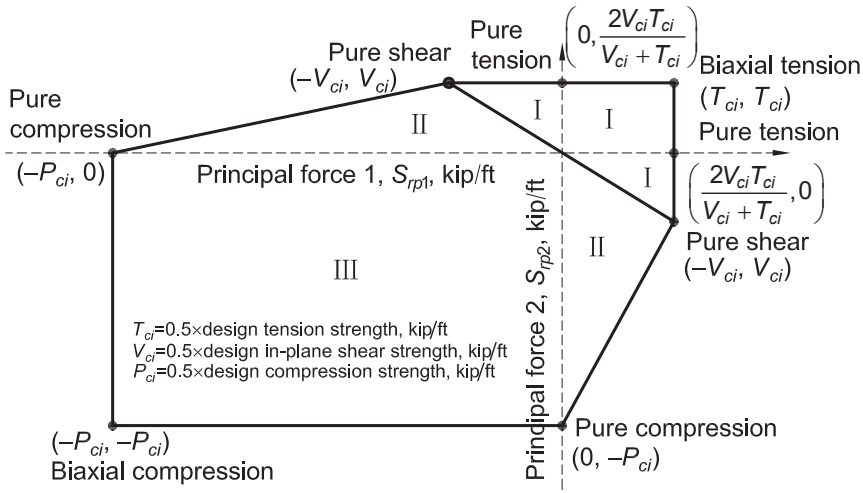


Fig. C-A-N9.3.9. Simplified interaction surface plotted in principal force space.

in-plane shear and biaxial tension in principal force space. This is always slightly conservative because (i) the pure in-plane shear strength $\left(V_{ci} = \kappa A_s F_y/2 \le A_s F_y/2\right)$ 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 $A_s F_y/2$.

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-N9.3.10 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-N9-34 to A-N9-36 were obtained by recasting the interaction Equations A-N9-27 to A-N9-29 (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.

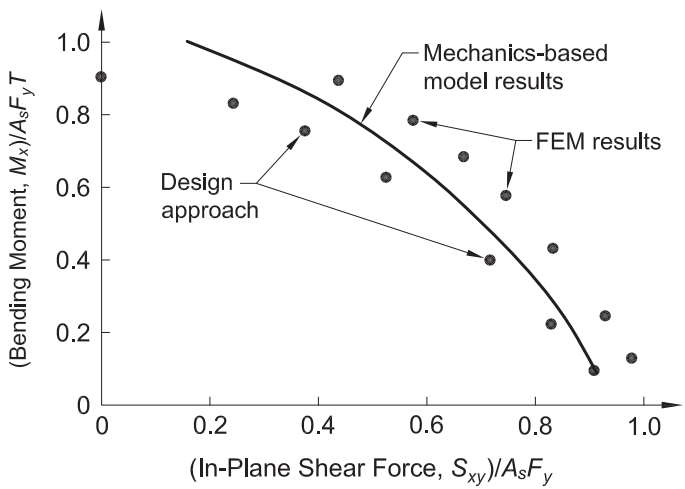


Fig. C-A-N9.3.10. Moment-shear interaction for SC structural element (Varma et al., 2014).

For example, Figure C-A-N9.3.11 shows the interaction surface defined by the interaction Equations A-N9-27 to A-N9-29 in terms of the principal forces, and some data points that were obtained using the alternate forms of the interaction Equations A-N9-34 to A-N9-36, which confirms their equivalency. Figure C-A-N9.3.11 was developed using 0.5-in.- (13-mm-) thick faceplates made from 50-ksi (345-MPa) yield stress steel filled with 29 in. (725 mm) of 6-ksi (40-MPa) concrete to develop a 30-in.- (750-mm-) thick SC structural element panel section. The anchor points in Figure C-A-N9.3.11 are without phi factors.

N9.4. DESIGN OF SC STRUCTURAL ELEMENT CONNECTIONS

1. General Provisions

The following connection types are possible: SC element-to-SC or -reinforced concrete (-RC) wall, SC element-to-RC basemat, SC element-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.

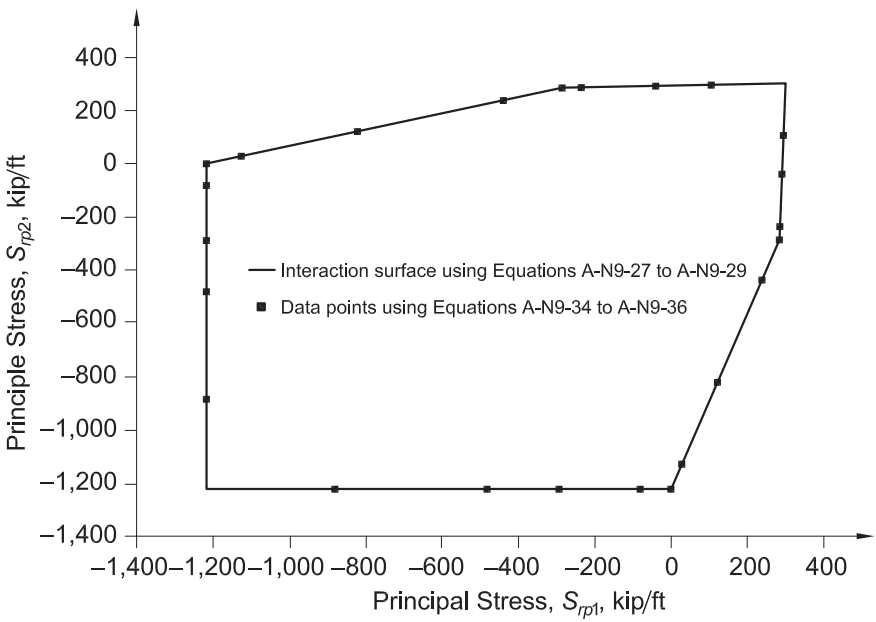


Fig. C-A-N9.3.11. Interaction surface and data points using alternate forms of interaction equations.

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, 2022d)]. 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. 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 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 composite members as multiple types of demands exist on plate/shell type SC elements. Unlike RC walls, SC structural elements 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 structural element as one unit.

1a. Required Strength

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

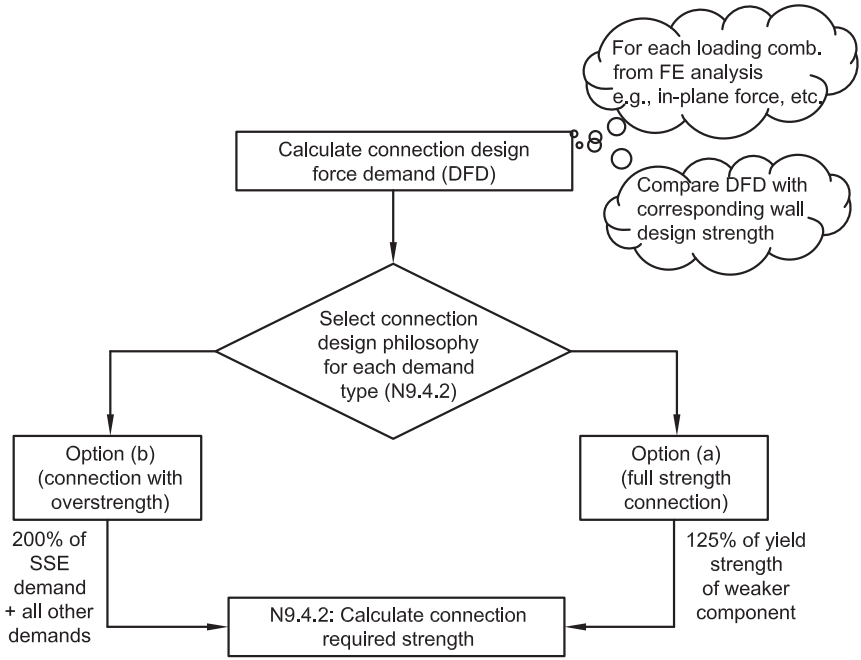


Fig. C-A-N9.4.1. Calculation of connection required strength.

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 structural elements. Because a full-strength connection is designed for 1.25 times the nominal strength of the weaker of the connected SC structural elements, the connection is always adequate, provided that the weaker SC structural element 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.

1b. 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.

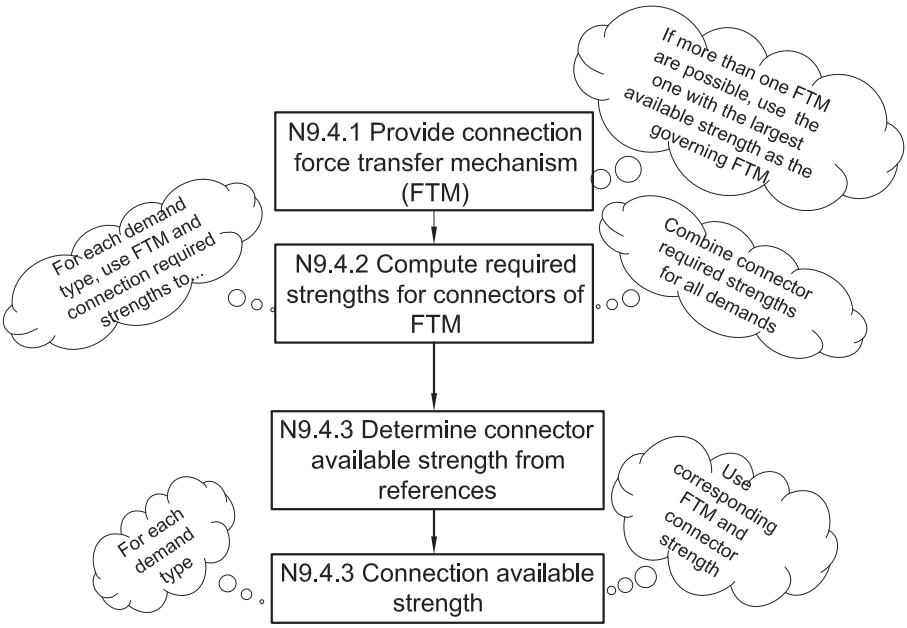


Fig. C-A-N9.4.2. Calculation of connection available strength.

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 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.

Seo and Varma (2017a) conducted an experimental study to investigate the joint shear behavior and joint shear strength of SC structural element-to-structural element L-joints. Based on the experimental results, the authors concluded that the SC L-joints can be designed using the full-strength connection philosophy and the ACI 349-06 (ACI, 2006) code equation can be used to estimate the joint shear strength, V_n^{js}, of SC structural element-to-structural element L-joints with γ of 8.

$$V_n^{js} = \gamma \sqrt{f_c'} A_j \qquad \text{(C-A-N9-18)}$$

where

A_j = effective cross-sectional area within joint, in.^{2}
f_c' = specified concrete compressive strength of concrete, psi
γ = parameter for joint shear strength

Seo and Varma (2019) conducted an experimental study to investigate the joint shear behavior and joint shear strength of SC structural element-to-structural element T-joints. The test parameters included the shear reinforcement ratio, ρ_t, and stud anchor layout in the joint region. The authors also evaluated the applicability of the

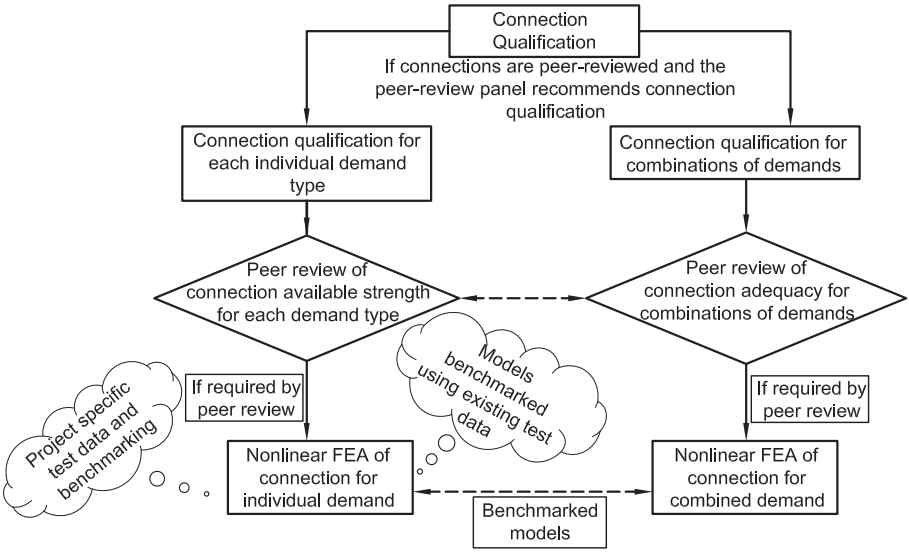


Fig. C-A-N9.4.3. Connection qualification.

equation for RC beam-column joint strength given in Section 21.5.3 of ACI 349-06 for estimating the joint shear strength of SC structural element-to-structural element T-joints. In the equation, the value of γ of 12 for RC beam-column joints [Case B of Type 2 in ACI 352R-02 (ACI, 2002)] with one column framing into the joint was selected for SC structural element-to-structural element T-joints. The effective cross-sectional area within the joint, A_j, is calculated using the effective joint widths and depths. For the SC structural element-to-structural element joints, A_j is calculated as the total cross-sectional area of the concrete infill within the joint region subjected to horizontal or vertical shear. Based on the experimental results, the authors concluded that (i) the effects of the shear reinforcement ratio, ρ_t, and stud anchor layout in the joint region are not significant and (ii) the ACI 349-06 code equation can be used to estimate the joint shear strength of SC structural element-to-structural element T-joints with γ equal to 12.

2. Lap Splicing of Reinforcing Bars with Faceplates

The prescriptive requirements for splices between SC elements and reinforced-concrete (RC) walls sections are based on the experimental investigations (Seo and Varma, 2017b; Seo et al., 2021). The dowel size limit is adopted from ACI 318-19 and ACI 318M-19 (ACI, 2019). Additional requirements associated with the dowel embedment length and location are to ensure forming and anchoring concrete compressive struts within the SC splice region adequately as illustrated in Figure C-A-N9.4.4. The interfacial strength requirement is to ensure transferring of tension from the dowels to the steel faceplate prior to rupturing of stud anchors.

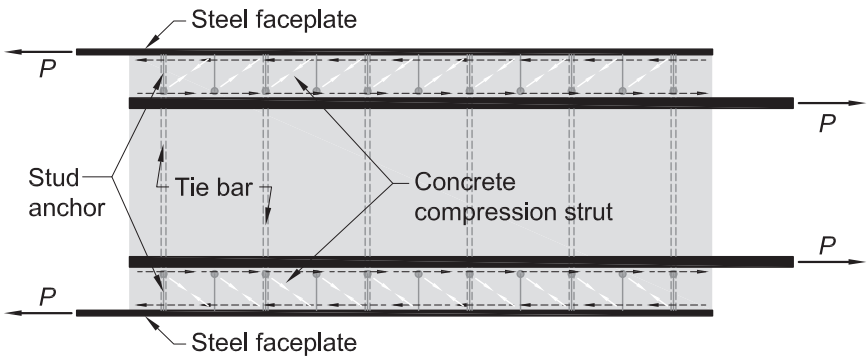


Fig. C-A-N9.4.4. Concrete compression struts in SC splices.

Add the following Appendix to the Specification Commentary.

APPENDIX N10

SPECIAL PROVISIONS FOR IMPACTIVE AND IMPULSIVE LOADS

This appendix applies to structural steel elements and steel-plate composite (SC) structural elements subject to impactive and impulsive loads.

Impactive loads include the following examples/types:

- tornado-borne missiles
- whipping pipes
- aircraft missiles
- other internal and external missiles

Impulsive loads include the following examples/types:

- jet impingement loads
- external blast pressure
- compartment pressurization
- jet shield reactions

Evaluation for beyond-design-basis aircraft missiles is outside the scope of this appendix. It is addressed in NEI 07-13 (NEI, 2011).

N10.1. GENERAL PROVISIONS

Section N10.1.1 invokes the stringent Charpy V-notch (CVN) requirements in Sections NA3.1 and NJ2.6 because high fracture toughness is deemed essential for ensuring the desired ductile performance against impulsive and impactive loads.

See Commentary NJ3.14 for discussion related to bolted and threaded parts used for connections of elements subject to impulsive or impactive loads.

The Dynamic Increase Factor (DIF) values in Section N10.1.2 reflect the fact that structural steels and concrete exhibit elevated strengths under high strain rate (such as due to impulsive or impactive loads), while the modulus of elasticity remains nearly constant. The DIF values specified in this section are based on the strain rate range that is representative of the impactive or impulsive loads for nuclear facilities. The values have been specified somewhat conservatively, which ensures some design margin. However, higher DIF values may be used if supported by test data and if the strain rate due to the impulsive or impactive load action is rigorously determined.

The DIF-based strength increases are permitted in other standards as well. ACI 349-13 and ACI 349M-13, 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, 2007).

Use of DIF values is not permitted for elastic behavior when the calculated dynamic load factor (DLF) is smaller than 1.20. This is because the accompanying strain rate is accordingly small for such low DLF value situations.

The concurrent presence of other loads (e.g., gravity loads) especially affects the post-elastic performance of structural elements subjected to impulsive or impactive loading. Accordingly, Section N10.1.3 requires that all concurrently acting loads, including the effect of loading sequence, be accounted for.

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

The width-to-thickness (b/t) ratios for elements resisting impulsive/impactive loads through flexure or compression are required to conform to the limits in Table A-N10.2.1. This table is conservatively adopted from similar requirements in Table D1.1 of the *Seismic Provisions* (ACI, 2022b) for various types of compression elements that are typically used for impulsive and impactive load applications in nuclear facilities. The limiting width-to-thickness ratios are based on treating the elements as highly ductile structural elements, which helps ensure that they will be able to undergo large strains/deformations without being limited by local buckling.

Table A-N10.2.1 generically refers to structural steel, and it is intended to be applicable to both carbon steel shapes and stainless steel shapes. It is true that the b/t limits in the table are based on carbon steel shapes, which have been adopted from the corresponding table in the *Seismic Provisions* for seismically compact applications involving carbon steel shapes. A comparison of the limiting width-to-thickness ratio (compact/noncompact) λ_p values from the *Specification* (AISC, 2022a) and ANSI/AISC 370-21 (AISC, 2021) indicates that the values for stainless steel shapes are always slightly larger than the corresponding values for carbon steel shapes. Accordingly, the seismically compact b/t limits from the *Seismic Provisions* that are meant for carbon steel shapes can be conservatively adopted for stainless steel shapes as well.

Section N10.2.4 provides acceptance criteria that are based on inelastic response of structural steel and composite members subjected to impactive and impulsive loads. The load effects (e.g., strains or displacements) due to impactive or impulsive forces are determined by inelastic analysis unless the behavior of affected elements remains elastic. As specified in this section, and as explained in the following, two analysis methods are permitted, each having their associated acceptance criteria.

If the dynamic load effect is determined by modeling the target as a single-degree-of-freedom (SDOF) elastic-plastic system, then the analysis can be performed using an idealized bilinear (or multilinear) load-deflection curve (sometimes referred to as the structural element's "resistance function"). For bilinear behavior,

the effective yield point for the idealized 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). For a multi-linear resistance function, the procedures described in UFC 3-340-02 (DOD, 2008) may be used. The required ductility ratio for the equivalent SDOF system is either determined by a nonlinear time-history analysis, or for well-defined impulse functions (e.g., for rectangular and triangular pulses), it can be directly determined using established response charts, such as those in Biggs (1964) and UFC 3-340-02. Alternatively, in the case of very short-duration impactive loads arising from essentially rigid/non-deformable missiles, 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 structural element by undergoing inelastic deformation. In either case, the element adequacy for the resulting inelastic deformation is verified by comparing the calculated required ductility ratio with the applicable permissible ductility ratio provided in Table A-N10.2.2. Note that Table A-N10.2.2 does not presently address composite structural elements. Composite structural elements are not commonly used in nuclear structures, especially for those structural elements resisting impactive or impulsive loads.

As an alternative, 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 element's adequacy is verified by ensuring that its calculated maximum plastic strain is less than or equal to 3%, which is a conservative strain limit for the steel grades that are permitted under this specification. The strain limit is permitted to be 5% for steel plate targets because of their 2D continuum and the attendant ability to better distribute the spread of inelastic response region. 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 element's nonlinear inelastic behavior.

Because the design is to be based on the element's own ductility (rather than that of its connections), use of Table A-N10.2.2 requires that the connection strength be 1.30 times that of the structural element's nominal strength. The 1.30 factor is based on the ratio of the expected yield stress to the minimum specified yield stress, R_y, values in the *Seismic Provisions* Table A3.1, with the understanding that materials with high yield strength variability (e.g., ASTM A36/A36M and ASTM A53/A53M) will not be used for applications involving impulsive and impactive loads.

The permissible ductility ratios in Table A-N10.2.2 are based on the following considerations:

(a) Axial Tension: Steel structural elements 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 structural elements. 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 (i.e., specified minimum yield stress to elastic buckling stress ratio, $F_y/F_e < 0.0225$) 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 the ratio of the strain corresponding to the onset of strain hardenting to the strain corresponding to nominal yield stress, $\varepsilon_{st}/\varepsilon_y$. 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 ductility factor, $\mu = 10$ 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.

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

This section provides SC-specific deformation/strain limits for regional/global response. An acceptable method for local response evaluation is discussed in the following.

2. Local Response Evaluation

The minimum required perforation thickness for SC structural elements subjected to impactive loads can be determined using project-specific test data or applicable published analytical methods developed from validated test data. In lieu of specific test data or published methods, the minimum required faceplate thickness (for a given value of concrete infill thickness) can be determined using Equation A-N10-2 (Kim, 2018). See Seo and Varma (2021) for an spreadsheet program for the implementation of the method developed by Kim (2018). Concrete penetration depth is calculated used the modified NDRC formula.

For cases where the perforation velocity, V_p, is greater than the initial velocity, V_i, the missile does not perforate through the concrete thickness and no steel plate thickness is required. The Section N10.3.2 equations, if used in this case, will result in a negative steel plate thickness value. The equations in Section N10.3.2 are applicable only to nondeformable missiles. For deformable missiles, the methodology proved by Bruhl et al. (2015) may be used. For large-diameter deformable missiles, a reduction factor, α_c, may be used for calculation of concrete penetration depth, x_c (NEI, 2011).

3. Special Design and Detailing Requirements

Presence of large opening(s) or small opening(s) with free edge at the opening perimeter can result in reduction of strength and inelastic deformation capacity of an SC structural element subjected to impulsive or impactive loads. Accordingly, refined analysis is required for such situations. Where practical, it is better to provide an independent impact barrier structure when such a situation is present.

Fracture limit state, which is non-ductile, is undesirable in the tension faceplate of SC structural elements subjected to impulsive or impactive loads. Accordingly, for bolted attachments, the associated bolt holes in tension faceplate need to be spaced sufficiently apart such that the fracture limit state does not govern. Similarly, welded attachments on the tension faceplate are to be avoided because they can cause a notch effect due to arc strikes, gouges, or defective workmanship, etc., which in turn can lead to premature fracture failure. These attachment related restrictions are similar to those for the designated protected zones in the *Seismic Provisions* (AISC, 2022b).

A properly designed and detailed flexure-controlled SC structural element is expected to undergo significant inelastic deformation when subjected to impulsive or impactive loading. This performance needs to be ensured by preventing the element's out-of-plane failure mode, which only provides limited ductility even in case of yielding shear reinforcement (ties), from occurring before faceplate yielding through flexure. Additionally, it is necessary to provide yielding shear ties because any unexpectedly yielded ties can still help sustain an extended ductile response regimen through continued flexural yielding of the faceplates that would precede yielding of the ties.

4. Analysis Requirements for Verification of Structural Element Ductility

This section provides acceptance criteria that are based on inelastic response of SC structural elements subjected to impulsive or impactive loads. The load effects (e.g., strains or displacements) due to impactive or impulsive forces are determined by inelastic analysis unless the behavior of affected elements remains elastic. As specified in this section, and as explained in the following, two analysis methods are permitted, each having their associated acceptance criteria.

If the dynamic load effect is determined by modeling the target as a single-degree-of-freedom (SDOF) elastic-plastic system, then the analysis can be performed using an idealized bilinear (or multilinear) load-deflection curve (sometimes referred to as the structural element's "resistance function"). For bilinear behavior, the effective yield point for the idealized 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). For multi-linear resistance function, the procedures described in UFC 3-340-02 (DOD, 2008) may be used. The required ductility ratio for the equivalent SDOF system is either determined by a nonlinear time-history analysis or, for well-defined impulse functions (e.g., for rectangular and triangular pulses), it can be directly determined using established response charts, such as those in Biggs (1964) and UFC 3-340-02. Alternatively,

in the case of very short-duration impactive loads arising from essentially rigid/non-deformable missiles, 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 structural element by undergoing inelastic deformation. In either case, the element adequacy for the resulting inelastic deformation is verified by ensuring that the chord rotation at supports does not exceed 6 degrees (0.105 radians). This is a conservative rotation limit since work by Bruhl and Varma (2018) indicates that SC slabs/walls can withstand rotations in excess of 10 degrees (0.175 radians). It is noted that SC slabs and walls in nuclear applications experience very small compressive/tensile membrane forces due to gravity loads and blast/pressurization type loads (e.g., those experienced by cross-walls in compartment pressurization situation). These small membrane forces have insignificant impact on the yield line behavior of SC walls/slabs in terms of their capacity to withstand pressures and their plastic deformation capacity.

As an alternative, the inelastic analysis can be based on use of a refined inelastic analysis that that uses a more rigorously determined stress-strain (or load deflection) curve. Under this approach, the element's adequacy is verified by ensuring that its calculated maximum plastic strain in faceplates is less than or equal to 5%. The 5% strain limit is considered conservative for the ductile steel material types that are required to be used for faceplates. Additionally, because the plastic neutral axis at plastic hinge locations is located immediately below the compression faceplate, the corresponding compression strain will be about one-tenth of the 5% limit, which is acceptable for the surrounding concrete in compression as it remains confined by the faceplates. (This provision thus obviates the need for checking concrete compressive strains.) Limited yielding of ductile ties is possible and acceptable at locations of high interfacial and/or out-of-plane shear demands. A 0.5% strain limit (i.e., 10% of that permitted for the faceplates) has been specified for the ties in order to ensure that bulk of the energy dissipation occurs from flexural yielding without jeopardizing a tie-related failure. A somewhat higher strain limit is permitted for SC structural elements (compared to the 3% strain limit for linear structural steel elements) because of their 2D continuum and the attendant ability to better distribute the spread of inelastic response region. 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 element's nonlinear inelastic behavior.

The SC connection design requirements in Section N9.4 are already robust enough to ensure that the failure will not be governed by connections.

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