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October 6, 2021 Docket No. 99902078

U.S. Nuclear Regulatory Commission
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SUBJECT: NuScale Power, LLC Submittal of Topical Report "Building Design and Analysis Methodology for Safety-Related Structures," TR-0920-71621, Revision 1

REFERENCE: Letter to NRC from NuScale, "NuScale Power, LLC Submittal of Topical Report 'Building Design and Analysis Methodology for Safety-Related Structures,' TR-0920-71621, Revision 0," dated December 18, 2020 (ML20353405)

NuScale Power, LLC (NuScale) hereby submits Revision 1 of the "Building Design and Analysis Methodology for Safety-Related Structures" (TR-0920-71621). The purpose of this submittal is to request that the NRC review and approve the building design and analysis methodology as described in the topical report. The acceptance review was completed for Revision 0 (reference). NuScale respectfully requests that the NRC's approval of Revision 1 be completed within six months of the date of transmittal.

Enclosure 1 contains the proprietary version of the report entitled "Building Design and Analysis Methodology for Safety-Related Structures," TR-0920-71621, Revision 1. NuScale requests that the proprietary version be withheld from public disclosure in accordance with the requirements of 10 CFR § 2.390. The enclosed affidavit (Enclosure 3) supports this request. Enclosure 2 contains the nonproprietary version of the report.

This letter makes no regulatory commitments and no revisions to any existing regulatory commitments.

If you have questions, please contact Liz English at 541-452-7333 or at eenglish@nuscalepower.com.

Sincerely,

Mark W. Shaver

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Enclosure 1: "Building Design and Analysis Methodology for Safety-Related Structures," TR-0920-71621-P, Revision 1, proprietary version
Enclosure 2: "Building Design and Analysis Methodology for Safety-Related Structures," TR-0920-71621-NP, Revision 1, nonproprietary version
Enclosure 3: Affidavit of Mark W. Shaver, AF-107606

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Licensing Topical Report

Building Design and Analysis Methodology for Safety-Related Structures

October 2021

Revision 1

Docket: 99902078

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List of Affected Pages

Revision Number	Page Number	Explanation
0	All	Initial Issue
1	Table 1-2, Throughout Sections 4.0, 5.0, 6.0, 7.0, 8.0 and 10.0.	Incorporation of RAI 9860, RAI 9834 and RAI 9833 responses.
1	Section 10.0	Editorial changes: Deleted references 10.1.28, 10.1.31 and 10.1.30.

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Abstract

There are efficiencies to be gained from recent developments in the civil, structural and seismic design areas that can be applied to the evaluation of the new generation of small modular reactor designs currently being considered for certification.

This report offers advanced building design and analysis methodologies to be used in the evaluation of Seismic Category I and II structures, for applicability to the new generation of small modular reactor designs. It is intended, but not required, to be used in conjunction with the NuScale Topical Report TR-0118-58005, Improvements in Frequency Domain Soil-Structure-Fluid Interaction Analysis (Reference 10.1.26), for the evaluation of complex Seismic Category I and II structures. Implementation of the 'soil library' evaluation methodology of TR-0118-58005 is implied in the presented Civil-Structural Seismic Design Criteria.

In addition, this report defines methodologies for the application of steel-plate composite (SC) walls designed for various loads in accordance with AISC N690-18 Appendix N9. New reactor designs have adopted modular SC structures as one of the design features for safety-related structures.

An SC wall consists of two steel plates (faceplates) with structural concrete between them. The faceplates are anchored to the basemat or floors using steel anchors and connected to each other using ties. Steel-plate composite walls have high shear resistance and have recently been approved for use in safety-related nuclear facilities. Steel-plate composite walls allow for offsite fabrication, eliminating the need for construction formwork and reinforcing bars (rebar), while meeting strength, stability and seismic requirements, thereby increasing the efficiency and productivity of construction.

The report also clarifies methodologies for the interaction of SC walls with traditionally constructed reinforced concrete (RC) members such as basemats, slabs, and roofs.

The presented design methodologies are in compliance with the requirements of the American Institute of Steel Construction, ANSI/AISC N690-18, Specification for Safety-Related Steel Structures for Nuclear Facilities, and the Specification for Structural Steel Buildings, ANSI/AISC 360-16, as required by ANSI/AISC N690-18.

Executive Summary

The design and analysis of Seismic Category I and II structures for current light water reactor applications have generally applied traditional methodologies and assumptions. In recent years, industry advancements have improved the design and evaluation of complex Seismic Category I and II structures.

The Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: Light Water Reactor Edition (NUREG-0800), 2010 (SRP), refers to AISC N690 for the design and evaluation of safety-related structures. This report addresses recent industry advances in safety-related structure design and analysis.

This report offers advanced building design and analysis methodologies implementing new developments in the design and evaluation of complex Seismic Category I and II structures, for applicability to the new generation of small modular reactor designs. The report presents a new Civil, Structural and Seismic Design Criteria intended, but not required, to be used in conjunction with the NuScale Topical Report TR-0118-58005, “Improvements in Frequency Domain Soil-Structure-Fluid Interaction Analysis” (Reference 10.1.26) evaluation methodology, also referred to as the 'soil library' seismic method in this report. The seismic analysis implements the soil library methodology for evaluation of complex structures. Seismic analyses are performed in conjunction with the soil library seismic method which includes dynamic impedance and load vectors for a soil substructure computed at different frequencies and different soil layered properties.

Additionally, the report provides methodologies for the application of steel-plate composite (SC) walls designed in accordance with AISC N690-18 Appendix N9. New reactors have adopted modular SC structures as one of the design features for safety-related structures.

The proposed methodologies presented in this report apply to a representative Reactor Building (RXB), Control Building (CRB), and Radioactive Waste Building (RWB). These structures are comprised of thick reinforced concrete basemats, walls, and slabs. Interior and exterior walls that are greater than 18-in. thick are typically designed using SC walls, whereas basemats, slabs, and roof members are reinforced concrete. Roof design may incorporate composite steel girders which are beyond the scope of this report.

The methodologies provide requirements and design parameters for designing and detailing the SC walls and connections. The report further describes the methodologies for reinforced concrete structures used in conjunction with the SC walls.

1.0 Introduction

1.1 Purpose

The purpose of this Topical Report is to present the methodologies to be used in building design and analysis for Seismic Category I and II concrete structures. Although these methodologies are intended to be used in conjunction with the Topical Report TR-0118-58005, “Improvements in Frequency Domain Soil-Structure-Fluid Interaction Analysis,” (Reference 10.1.26), such methodologies can be used with a commercially-available analysis program that can perform seismic analyses of a complex safety-related structure.

The results of this analytical procedure provide design parameters for the seismic qualification of nuclear safety-related structures, systems, and components (SSC).

1.2 Scope

The building design and analysis methodology discussed in this report applies to Seismic Category I and II reinforced concrete and steel-plate composite structure designs in nuclear reactor facilities. Although presented in the context of use with a representative small modular reactor (SMR) application, the report scope is not necessarily limited to a specific design.

The report offers, in order of presentation, an advanced in-structure response spectra (ISRS) and design methodology for Seismic Category I and II SSC (Section 4.0), determination of effective stiffness for Seismic Category I and II structures analysis (Section 5.0), a design methodology for steel-plate composite (SC) walls (Section 6.0), a design methodology for SC wall connections (Section 7.0), and design methodology for Seismic Category I and II concrete structures (Section 8.0).

1.3 Abbreviations and Definitions

The abbreviations used in this report are shown in Table 1-1. Definitions of common terms used in the report are shown in Table 1-2. Symbols are presented in Table 1-3.

Table 1-1 Abbreviations

Term	Definition
ASME	American Society of Mechanical Engineers
BPVC	Boiler and Pressure Vessel Code
CAP	Capitola (station name of strong ground motion recording)
CHI	Chi-Chi (station name of strong ground motion recording)
CJP	complete-joint-penetration
CNV	containment vessel
CRB	Control Building
CSDRS	certified seismic design response spectra
CSDRS-HF	certified seismic design response spectra - high frequency
DCR	demand-to-capacity ratio

Table 1-1 Abbreviations (Continued)

Term	Definition
DOF	degrees-of-freedom
ELC	El Centro (station name of strong ground motion recording)
FEA	finite element analysis
GDC	General Design Criteria
ISRS	in-structure response spectra
IZM	Izmit (station name of strong ground motion recording)
LCN	Lucerne (station name of strong ground motion recording)
NRC	Nuclear Regulatory Commission
RC	reinforced concrete
RL	response level
RWB	Radioactive Waste Building
RXB	Reactor Building
SC	steel-plate composite
SMR	small modular reactor
SSC	structures, systems, and components
SSE	safe shutdown earthquake
TRB	triple building
YER	Yermo (station name of strong ground motion recording)

Table 1-2 Definitions

Term	Definition
beam-column member	A structural member that supports applied loads and its own weight primarily by internal moments, shears, and axial loads. Floor beams are examples of beam-column members.
boundary elements	Portions along structural wall and structural diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms can also be provided with boundary elements.
building substructure	Model representing the building mass and stiffness.
collectors	Tension and compression members that gather (collect) shear forces from diaphragms and deliver them to the vertical members.
control point	Location where the seismic input is defined. Typically the surface of the free field or the free-field layer at the elevation of the building foundation.
equipment substructure	Model representing the mass and stiffness of a particular component. There can be multiple equipment substructures that interact with each other, with fluid substructures, and with the building substructure.
floor beam	A beam-column member that is cast together with the slab.
fluid substructure	Model representing the mass, viscous damping, stiffness, and fluid-structure coupling terms of the acoustic fluid elements. There can be multiple fluid substructures that interact with each other, with equipment substructures, and with the building substructure.
in-plane actions	Forces and moments acting along the plane of structural walls and structural diaphragms, causing in-plane moments, shears, and axial loads.

Table 1-2 Definitions (Continued)

Term	Definition
isotropic material	a material having identical property values in all directions. In particular, the Young's modulus and the shear modulus are related by Hooke's law for isotropic materials.
lateral-force resisting system	That portion of the structure comprised of members proportioned and detailed to resist the design seismic forces.
limit state	The limiting acceptable condition of the structure. The limit state can be defined in terms of a maximum acceptable displacement, strain, ductility, or stress.
load path	The path of resistance consisting of structural members that the imposed load follows from the point of origin (inertial forces at location of structure mass) to the point of final resistance (i.e., supporting soil).
one-way slab	Slab supported on two opposite sides only, in which case the structural action of the slab is essentially one-way, the loads being carried by the slab in the direction perpendicular to the supporting members. One-way action also occurs in slabs supported in all four sides, if the ratio of length to width of one slab panel is larger than 2. In this case, most of the load is carried in the short direction to the supporting members.
orthotropic material	a material having properties that differ along three mutually-orthogonal axes. The Young's modulus and shear modulus are independently defined along the three axes.
out-of-plane actions	Forces and moments acting perpendicular to the plane of slab or wall panels, causing out-of-plane moment and shears.
reinforced concrete slab	A broad, flat plate, usually horizontal, that carries loads perpendicular to its plane. It can be supported by RC beams, walls or columns. In this document, they are simply referred to as slabs.
safe shutdown earthquake	Earthquake that produces the vibratory ground motion for which certain structures, systems, and components (SSC) in the nuclear power plant must be designed to remain functional.
Seismic Category	Nuclear safety-related classification assigned to SSC based on seismic hazard and functionality demand.
Seismic Category I	Structures, systems and components designed to withstand the seismic loads associated with the SSE, in combination with other designated loads, without loss of function or pressure integrity.
Seismic Category II	Structures, systems, and components that perform no safety-related function, but whose structural failure or adverse interaction could degrade the functioning or integrity of a Seismic Category I SSC to an unacceptable level or could result in incapacitating injury to occupants of the control room during or following an SSE, are designed and constructed so that the SSE would not cause such failure.
slab panel	Portion of slabs bounded by floor beams or walls.
soil impedance	The dynamic stiffness of soil supporting a foundation. Impedance includes stiffness, inertial effects, radiation damping, and material damping. Soil impedance is usually frequency dependent and has a complex value.
soil library	A collection of soil impedance and seismic load vectors for a soil substructure computed at different frequencies and different soil layered halfspace properties.
soil substructure	Model representing the soil impedance of a layered elastic halfspace with an excavation.
steel-plate composite wall	An SC wall consists of two steel plates (faceplates) composite with structural concrete between them, where the faceplates are anchored to concrete using steel anchors and connected to each other using ties.

Table 1-2 Definitions (Continued)

Term	Definition
structural diaphragms	Structural members, such as floor and roof slabs, that transmit inertial forces to vertical elements of the lateral force-resisting members.
substructure	Submodel that can be combined with other submodels. The term substructure does not imply that an ANSYS superelement is utilized.
tension and compression chord	Boundary elements in structural diaphragms that resist in-plane moments by tension or compression forces.
transfer function	Mathematical relationship that describes the response of one part of a structure to a given input at a control point.
two-way slab	Slab supported in all four sides. Two-way action is obtained if the ratio of length to width of the slab panel is not larger than 2.

Table 1-3 List of symbols

Symbol	Definition
A_b	The nominal unthreaded body of bolt or threaded part (in.^{2})
A_{bm}	The cross sectional area of the base metal (in.^{2})
A_{brg}	Net bearing area of the head of stud, anchor bolt, or headed deformed bar (in.^{2})
A_c	Area of concrete infill per unit width (in.^{2}/ft)
A_{cm}	Missile contact area (in.^{2})
A_g	The gross area of the base metal (in.^{2})
A_{gv}	The gross area of the base metal subject to shear (in.^{2})
An	The net area of tension for the base metal (in.^{2})
A_{Nc}	Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension (in.^{2})
A_{Nco}	Projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing (in.^{2})
A_{ntie}	Net sectional area of the tie (in.^{2})
A_{nv}	The net area of the base metal subject to shear (in.^{2})
A_s	Gross area of faceplates per unit width (in.^{2}/ft)
Asa	Cross-sectional area of steel-headed stud anchor (in.^{2})
A_{sa}	Total cross-sectional area of a steel-headed stud anchors (in.^{2})
A_{se}	The effective cross-sectional area of a single anchor in shear (in.^{2})
As^F	Gross cross-sectional area of faceplate in tension due to flexure, per unit width (in.^{2}/ft)
Asn	Net area of faceplates per unit width (in.^{2}/ft)
A_{sr}	Cross-sectional area of the anchor reinforcement (in.^{2})
$Atie$	Gross sectional area of the tie (in.^{2})
A_{Vc}	Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear (in.^{2})

Table 1-3 List of symbols (Continued)

Symbol	Definition
A_{Vco}	Projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness (in.^{2})
A_{we}	The effective area of the weld (in.^{2})
b	Largest unsupported length of the faceplate between rows of steel anchors or ties. The value of b is to be calculated as the minimum spacing between ties or anchors in the horizontal and vertical directions of the faceplate (in.).
C	Rated capacity of crane
$c1$	Factor used to determine spacing of steel anchors
$c2$	Calibration constant for determining effective flexural stiffness
$c_{a,\ min}$	Minimum distance from center of an anchor shaft to the edge of concrete (in.)
c_{a1}	Distance from the center of an anchor shaft to the edge of concrete in one direction (in.). If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance.
c_{a2}	Largest edge distance of a headed stud anchor (in.)
C_{cr}	Rated capacity of crane, including the maximum wheel loads of the crane and the vertical, lateral, and longitudinal forces induced by the moving crane, including impact factor
C_m	Specific heat used in the elastic finite element model of SC panel section (Btu/lb*D*°F.)
C_v	Specific heat of the concrete (Btu/lb°F.)
d	Distance from extreme compression fiber to centroid of longitudinal tension reinforcement.
D	Average outer diameter of the missile (in.)
D	Dead loads, including piping, equipment, partitions, and dead loads of the crane components
D_L	Dead loads due to the weight of the structural elements, fixed-position equipment, and other permanent fixtures
D_m	Maximum displacement from the analysis (in.)
d_m	Missile equivalent diameter (in.)
d_s	Stud diameter (in.)
d_{sh}	Stud head diameter (in.)
D_y	Effective yield displacement (in.)
E_c	Modulus of elasticity of concrete $w_c 1.5 \sqrt{f_c'}$ (ksi)
EI_{eff}	Effective flexural stiffness for analysis of SC walls per unit width(kip-in.^{2}/ft)
$EI_{eff\text{-}buck}$	Effective SC stiffness per unit width used for buckling evaluation (kip-in.^{2}/ft)
E_m	Material elastic modulus used in the elastic finite element model (ksi)
e_N'	Distance between resultant tension load on a group of anchors loaded in tension and the centroid of the group of anchors loaded in tension (in.)
E_o	Load effects of operating basis earthquake, including operating-basis earthquake-induced piping and equipment reactions
Es	Modulus of elasticity of steel taken as 29,000 ksi for carbon steel and 28,000 ksi for stainless steel

Table 1-3 List of symbols (Continued)

Symbol	Definition
E_{ss}	Load effects of an SSE, including SSE-induced piping and equipment reactions
e_V'	Distance between resultant shear load on a group of anchors loaded in shear in the same direction, and the centroid of the group of anchors loaded in shear in the same direction (in.)
F	Loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights
f'_c	Specified compressive strength of concrete (ksi)
F'_{nt}	The nominal tensile stress modified to include the effect of shear stress (ksi)
F_{EXX}	The filler metal classification strength (ksi)
F_{nbm}	The nominal stress of the base metal of the weld (ksi)
F_{nr}	Nominal rupture strength of the tie or the nominal strength of the associated connection, whichever is smaller (kips)
F_{nt}	Nominal tensile stress of the bolt (ksi)
F_{nv}	Nominal shear stress of the bolt (ksi)
F_{nw}	The nominal stress of the weld metal (ksi)
Ft	Nominal yield strength of the tie, min(F_{ny}, F_{nr}) (ksi)
F_u	Specified minimum tensile strength (ksi)
F_{ua}	Specified minimum tensile strength of a steel-headed stud anchor (ksi)
f_{ur}	Specified minimum yield stress for the supplementary reinforcement (ksi)
f_{uta}	Specified tensile strength of anchor steel (ksi)
F_y	Specified minimum yield stress (ksi)
Fya	Specified minimum yield stress of the anchor (ksi)
Fyt	Nominal yield stress of the tie (ksi)
G	Shear modulus of elasticity of steel (11,200 ksi for carbon steel and 10,800 ksi for stainless steel)
GA_{eff}	Effective in-plane shear stiffness per unit width (kip/ft)
GA_{uncr}	In-plane shear stiffness of uncracked composite SC panel section per unit width (kip/ft)
Gc	Shear modulus of elasticity of concrete $772\sqrt{f_c'}$ (ksi)
H	Loads due to weight and pressure of soil, water in soil, or bulk materials
h_a	The thickness of member in which an anchor is located, measured parallel to anchor axis (in.)
h_{an}	Length from the base of the steel-headed stud anchor to the top of the stud head after installation (in.)
h_{ef}	The effective embedment length (in.)
I_c	Moment of inertia of concrete infill per unit width (in^4/ft)
I_s	Moment of inertia of faceplate corresponding to the cracked concrete case (in^4/ft)
j_x, j_y	Parameter for distributing required flexural strength into the force couple acting on each notional half of the SC panel section
j_{xy}	Parameter for distributing required twisting moment strength into the force couple acting on each notional half of the SC panel section
K	Effective length factor for buckling
k	Strength coefficient (related to missile impact)

Table 1-3 List of symbols (Continued)

Symbol	Definition
κ	Calibration constant for determining in-plane shear strength
k_{cp}	Coefficient for pryout strength
K_m	Thermal conductivity used in the elastic finite element
L	Unsupported length of the SC wall (in.)
l	Unit width 12 (in./ft)
L_d	Development length (in.)
L_L	Live load due to occupancy and moveable equipment, including impact (Section 6.4.1)
L_r	Roof live load
lu	Unsupported length of the SC wall taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support to the SC wall
L_w	The length of the weld (in.)
M	Mass of the missile
M_1	Small factored end moment on the SC wall, the value is taken as positive if the member is bent in single curvature and negative if bent in double curvature (in-lb)
M_2	Large factored end moment on the SC wall value of M_2 is always positive (in-lb)
$M_{r,th}$	Theoretical maximum out-of-plane moment per unit width induced due to thermal gradient (kip-in./ft)
M_{rx}, M_{ry}	Required out-of-plane flexural strength per unit width (kip-in./ft)
M_{rxy}	Required twisting moment strength per unit width (kip-in./ft)
n	Modular ratio of steel and concrete Es/Ec
n	Number of stud headed anchors
N	Missile shape factor
N_b	Basic concrete breakout strength in tension of a single anchor in cracked concrete (lb)
N_{cb}, N_{cbg}	Nominal concrete breakout strength in tension of a single anchor and a group of anchors (lb)
n_{es}	Effective number of steel anchors contributing to a unit cell
n_{et}	Effective number of ties contributing to a unit cell
N_p	Pullout strength in tension of a single anchor in cracked concrete (lb)
N_{pn}	Nominal pullout strength in tension of a single anchor (lb)
N_{sa}	Nominal strength of a single anchor or individual anchor in a group of anchors in tension as governed by the steel strength (lb)
N_{sb}, N_{sbg}	Side-face blowout strength of a single and a group of anchors respectively (lb)
Pa	Maximum differential pressure load generated by the postulated accident
Pci	Available compressive strength per unit width (kip/ft)
Pe	Elastic critical buckling load per unit width (kip/ft)
Pno	Nominal compressive strength per unit width (kip/ft)
Ps	Factor used to calculate the shear reinforcement contribution to out-of-plane shear strength
Qct	Available tensile strength of steel-headed stud anchors (kips)
Q_{cv}	Design shear strength of a steel-headed stud anchor (kips)
Qcv^{tie}	Available interfacial shear of the tie bars (kips)

Table 1-3 List of symbols (Continued)

Symbol	Definition
R	Rain load
r	Radius of gyration, taken as 0.30 t_{sc} for SC walls
R	The radius of joint surface (in.)
r_1	Minor radius of conical plug (in.)
r_2	Major radius of conical plug (in.)
R_a	Pipe and equipment reactions generated by the postulated accident, including R_o
R_o	Pipe reactions during normal operating, start-up, or shutdown conditions, based on the most critical transient or steady- state condition
S	Snow load as stipulated in ASCE/SEI 7 for Category IV facilities
s	The maximum spacing between anchors within the group (in.)
S_{cr}	Concrete cracking threshold per unit width (kip/ft)
S_e	Extreme snow load
S_{rx}, S_{ry}	Required membrane axial strength per unit width in directions x and y respectively (kip/ft)
$S'rx$, S'_{ry}	Required membrane axial strength per unit width in directions x and y respectively for each notional half of SC panel section (kip/ft)
S_{rxy}	Required membrane in-shear strength per unit width (kip/ft)
S_{rxy}'	Required membrane in-shear strength per unit width for each notional half of panel section (kip/ft)
S_t	Spacing between ties in the horizontal and vertical directions (in.)
S_{tl}	Spacing of shear reinforcement along the direction of one-way shear (in.)
S_{tt}	Spacing of shear reinforcement transverse to the direction of one-way shear (in.)
T_a	Thermal loads generated by the postulated accident
t_c	Concrete infill thickness (in.)
T_{ci}	Available tensile strength per unit width for each notional half of SC panel section
t_m	Model section thickness used in the elastic finite element
T_o	Thermal effects and loads during normal operating, start-up, or shutdown conditions, based on the most critical transient or steady state condition
T_p	Faceplate tensile strength per unit width
t_p	Faceplate thickness
t_{rp}	Required rear steel plate thickness for impact (in.)
t_{sc}	SC section thickness
$t_{th\text{-}eff}$	The effective throat of welds (in.)
V_b	Basic concrete breakout strength in shear of a single anchor in cracked concrete (lb)
Vc	Available out-of-plane shear strength per unit width of SC panel section (kip/ft)
V_{cb}, V_{cbg}	Nominal concrete breakout strength in shear of a single anchor or a group of anchors (ksi)
V_{ci}	Available in-plane shear strength per unit width for each notional half of the SC panel section (kip/ft)
V_{conc}	Nominal out-of-plane shear strength contributed by the concrete per unit width of the SC panel section (kip/ft)
V_{cp}	Nominal concrete pryout strength of a single anchor (lb)

Table 1-3 List of symbols (Continued)

Symbol	Definition
V_o	Initial impact velocity of the missile before impact (ft/sec)
V_p	Missile velocity that just initiates perforation (ft/sec)
V_r	Residual velocity (ft/sec)
V_{rx}, V_{ry}	Required out-of-plane shear per unit width along edge parallel to directions x and y respectively
V_s	Contribution of the steel shear reinforcement (ties) to the nominal out-of-plane shear strength per unit width of the SC panel section (kip/ft)
W	Wind load as stipulated in ASCE/SEI 7 for Category IV facilities
W_m	Total missile weight (lbs)
w_c	Weight of concrete per unit volume (lbs/ft^3)
Wcp	Weight of the concrete plug (lbs)
W_h	Hurricane load
Wt	Loads generated by the specified design tornado, including wind pressures, pressure differentials, and tornado-borne missiles
Y_j	Jet impingement load, or related internal moments and forces, on the structure generated by a postulated pipe break
Y_m	Missile impact load, such as pipe whipping 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
α	Ratio of available in-plane shear strength to available tensile strength for each notional half of the SC panel section
α_c	Thermal expansion coefficient of concrete (1/°F.)
α_m	Thermal expansion coefficient used in the elastic finite element analysis of SC panel section (1/°F.)
α_p	Perforation reduction factor
α_s	Thermal expansion coefficient of faceplate (1/°F.)
β	Ratio of available in-plane shear strength to available compressive strength for each notional half of the SC panel section The β statistical variation factor is discussed in Section 6.7
γ_m	Material density used in the elastic finite element analysis of the SC panel section (lb/in.^{3})
γ_s	Density of the steel (lb/in.^{3})
ΔT_{savg}	Average of the maximum surface temperature increases for the faceplates due to accident thermal conditions (°F.)
ΔT_{sg}	Maximum temperature differences between the faceplates due to accident thermal conditions (°F.)
θ	Conical plug angle (degrees)
μ_{DD}	Ductility ratio demand
ξ	Factor used to calculate the shear reinforcement contribution to out-of-plane shear strength
ρ	Reinforcement ratio
ρ'	Stiffness-adjusted reinforcement ratio
υ_c	Poisson's ratio of the concrete

Table 1-3 List of symbols (Continued)

Symbol	Definition
υ_m	Poisson's ratio used in the elastic finite element analysis of SC panel section
$\psi_{c,N}$	Factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete
$\psi_{c,P}$	Factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete
$\psi_{c,V}$	Factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement
$\psi_{ec,N}$	Factor used to modify tensile strength of anchors based on eccentricity of applied loads
$\psi_{ec,V}$	Factor used to modify shear strength of anchors based on eccentricity of applied loads
$\psi_{ed,N}$	Factor used to modify tensile strength of anchors based on proximity to edges of concrete member
$\psi_{ed,V}$	Factor used to modify shear strength of anchors based on proximity to edges of concrete member
$\psi_{h,V}$	Factor used to modify shear strength of anchors located in concrete members with $h_a < 1.5c_{a1}$
ϕ_t	Strength reduction factor for tension
ϕ_v	Strength reduction factor for shear
$\bar{\rho}$	Strength-adjusted reinforcement ratio

2.0 Background

Industry advancements have improved the design and evaluation of complex Seismic Category I and II structures, while facilitating constructability, increasing cost efficiencies, and shortening schedules for assembly.

This report offers advanced design and analysis methodologies implementing these new developments in the design and evaluation of complex Seismic Category I and II structures for applicability to the new generation of SMR designs.

Section 4.0 updates the methodology traditionally used for the evaluation of Seismic Category I and II structures. Section 4.1 presents a methodology for providing analytical models with damping values and stiffness properties based on the actual stress state of the members under seismic load. Section 4.2 describes the member design process for load combinations that involve seismic loads.

Section 5.0 describes a methodology for determining the effective stiffness of reinforced concrete (RC) members and SC walls for use in seismic analysis, including modeling approaches to represent effective stiffness for RC wall, RC slab members and for SC walls in Seismic Category I and II structures.

Section 6.0 discusses the design methodology for application of SC walls in lieu of RC walls. Section 7.0 addresses the methodology for SC wall connections. Both Section 6.0 and Section 7.0 are for use in the construction of Seismic Category I and II Structures.

Section 8.0 defines the analysis and design methodology for the RC members.

2.1 Regulatory Requirements

2.1.1 10 CFR 50 Appendix A GDC 50

In accordance with General Design Criteria (GDC) 50, nuclear power unit structures, systems, and components (SSC) important to safety are designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions.

3.0 Software

The following software is used in this methodology for seismic analysis and design of Seismic Category I and II SSC:

- ANSYS version 18.2.2

The following general purpose, utility, or software tools are used to prepare plots contained in this report and for calculations that are verified by alternate calculations or inspection:

- Python (Linux version 2.7.5 and Windows version 2.7.13)
- Mathcad version 15.1
- Microsoft Excel

4.0 In-Structure Response Spectra and Design Methodology of Member Forces for Seismic Category I and Seismic Category II Structures, Systems, and Components

This section describes the methodology used to obtain in-structure response spectra (ISRS) for subsystem design and member forces for design of Seismic Category I and II SSC. The goal of this methodology is to provide analytical models with damping values and stiffness properties based on the actual stress state of the members under the most critical seismic load combination.

The process is summarized in Figure 4-1 and includes the development of two ANSYS models. The ANSYS models comprise a representative Reactor Building (RXB), the Control Building (CRB), and the Radioactive Waste Building (RWB), surrounded by engineered backfill. These models are referred to as triple building (TRB) models.

The following TRB models are generated:

- **TRB Seismic**. This model is used for seismic analysis in conjunction with the soil library seismic method (Reference 10.1.26). It includes the effective seismic mass and the soil library (dynamic impedance and load vectors) for the evaluated soil types (7, 9, and 11). Member forces and ISRS corresponding to the safe shutdown earthquake (SSE) (E_{ss}) are obtained. Soil type 11 represents a soft soil profile, soil type 7 represents a rock soil profile, and type 9 represents a hard rock soil profile.
- **TRB Static**. This model is used to obtain the member forces from non-seismic loads, and includes the halfspace corresponding to soil type 11 with the stiffness reduced by half to account for settlement effects.

  In-column (in-layer) excitations are used in order to perform seismic analysis with the soil library seismic method (Reference 10.1.26). For the TRB seismic analysis, in-column excitations are applied at the base elevation of the RXB.

  Figure 4-1 shows the analysis process for three selected soil types (11, 7, and 9). Five certified seismic design response spectra (CSDRS)-compatible excitations act on soil types 11 and 7. The process starts with the TRB Seismic model set to {{ }}[2(a),(c)] stiffness properties for members. Then, the following main steps are performed for each soil type:

  - Determination of member effective stiffness (cracked or uncracked) and damping ratio for the seismic load cases
  - Force calculation for required load combinations and member design
  - ISRS calculation for subsystem design

Figure 4-1 Strategy for in-structure response spectra and member design

{{

}}2(a),(c)

(Note: Soil types 11 and 7 are analyzed with CSDRS-compatible excitations CAP, CHI, ELC, IZM, and YER. Soil type 9 is evaluated with CSDRS-HF-compatible excitation LCN.)

4.1 Determination of Effective Stiffness and Damping

During seismic analysis with the SSE, it is expected that some members crack while others remain uncracked. To determine whether a particular member is cracked or uncracked, representative stress levels for the various actions (i.e., in-plane shear) are investigated for the controlling load combination. An effective stiffness is then assigned whenever the representative stress exceeds the cracking limit of the material.

The procedure presented in this section applies to both RC members and SC walls. Effective stiffness values for concrete members and SC walls are included in ASCE 4-16 (Reference 10.1.2) and AISC N690-18 (Reference 10.1.3), respectively.

In agreement with ASCE 4-16 and AISC N690-18, walls and slabs subject to out-of-plane flexure are considered cracked; that is, an effective stiffness is used. For SC walls, this effective stiffness is further reduced for accident thermal conditions (Reference 10.1.3). Members experiencing out-of-plane shear are considered uncracked (i.e., gross stiffness is used); therefore, cracking is investigated only for in-plane shear and in-plane bending of walls and slabs.

Representative in-plane shear or in-plane bending stresses can be obtained by averaging the stresses across the entire member (i.e., entire wall or slab panels); however, this may not be in agreement with the member seismic behavior. For instance, for cantilever shear walls, the maximum shear and moment usually occur at the base of the wall. Thus, once the stresses corresponding to these forces exceed the cracking limits, the wall is considered cracked (i.e., it experiences a significant reduction in its lateral stiffness). Accordingly, in this methodology, cracking is evaluated at the following critical sections for walls: at the base and right above the ground level; and, for slabs: at the middle of the span and at the span ends (i.e., at slab-wall connections). The maximum stress at any of these locations is used to evaluate cracking for the member.

Damping values are used in linear elastic analysis and depend on the level of cracking expected during the SSE. According to ASCE 43-19 (Reference 10.1.1), the level of cracking is related to the response level (RL) of the seismic load-resisting members.

The RL is determined, on a member-by-member basis, based on the demand-to-capacity ratio (DCR), including seismic and non-seismic loads. Thus:

if DCR ≤ 0.5, RL 1 is used

if $0.5 <$ DCR < 1, RL 2 is used

if DCR ≥ 1, RL 3 is used

In this design methodology, limit state D is considered; thus, {{ }}[2(a),(c)].

Based on ASCE 4-16 and ASCE 43-19, RL 2 damping values are used for evaluating seismic-induced forces and moments in structural members. On the other hand, damping values based on the actual RL are used for generating ISRS for subsystem design.

The damping values corresponding to RL 1 and RL 2 are shown in Table 4-1 below, for different types of structures. For SC walls, AISC N690-18 specifies a maximum damping ratio of 5 percent for seismic analysis involving the SSE, which is considered RL 2. For RL 1, a damping ratio of 3 percent is used in this methodology based on ASCE 43-19 (Reference 10.1.1). The damping ratios for SC walls are provided in Table 4-1.

Table 4-1 Viscous damping expressed as a fraction of critical damping (adapted from ASCE 4-16)

Structure Type	**Response Level 1**	**Response Level 2**
Welded aluminum structures	0.02	0.04
Welded and friction-bolted steel structures	0.02	0.04
Bearing-bolted steel structures	0.04	0.07
Prestressed concrete structures (without complete loss of prestress)	0.02	0.05
RC structures	0.04	0.07
Reinforced masonry shear walls	0.04	0.07
SC walls(1)	0.03	0.05

(1) SC Walls damping values are not based on ASCE 4-16

ASCE 4-16 adds to the definition of RL based on the cracking state of shear walls. When demands in shear-critical walls are less than $3\sqrt{f_c'}$ (psi), RL 1 is considered. For walls in which demands are greater than $3\sqrt{f_c'}$ (psi), RL 2 is used.

From ASCE 4-16 and ASCE 43-19 guidelines, the effective stiffness and RL in this methodology are both determined based on the in-plane cracking state of the members that constitute the seismic load-resisting system. Thus, if a member is cracked for the controlling seismic load combination, {{ }}2(a),(c) is assigned.

If the member is uncracked, {{ }}2(a),(c) is used. For ISRS generation, a damping ratio is selected from Table 4-1 and assigned to each individual member. Finally, in-plane effective stiffness is assigned to each individual member according to its cracking state.

The cracking limits for concrete members and SC wall sections transformed to concrete are derived as shown below:

In-plane bending $f_{cr} = 7.5\sqrt{f_c'}$ *(psi)*

In-plane shear $\upsilon_{cr} = 3\sqrt{f_c'}$ *(psi)*

Where f_c' is the specified concrete compressive strength (psi).

In-plane shear and in-plane bending cracking and damping are evaluated only in the seismic-load-resisting-system members because these members have a major effect on the building response. The in-plane shear and in-plane bending stresses in members not part of the lateral load resisting system are minimal; therefore, they are considered uncracked and assigned {{ }}[2(a),(c)] damping values for ISRS generation. As explained above, damping values corresponding to {{ }}[2(a),(c)] are used for member design.

For the representative RXB, CRB, and RWB, the main lateral-force-resisting systems are comprised of shear walls located along the building perimeter (aligned to both main horizontal directions), the roof, and main floors acting as diaphragms. For the RXB, there are also internal shear walls, as well as the pool walls. Thus, cracking is evaluated in those members.

The controlling load combination for cracking evaluation is determined in Section 4.1.1. The process for assigning effective stiffness and damping ratios is explained in Section 4.1.2.

4.1.1 Controlling Load Combination for In-Plane Cracking Evaluation

In accordance with ASCE 4-16, in-plane cracking is evaluated considering the most critical seismic load combination. For load combinations not involving the SSE, uncracked in-plane stiffness is used for members.

There are two load combinations involving seismic loads. For concrete structures, these are load combinations (9-6) and (9-9) in ACI 349-13 (Reference 10.1.4). For steel structures, these are load combinations (NB2-6) and (NB2-9) in AISC N690-18 (Reference 10.1.3). From this point forward in this topical report, "ACI 349" will be used to reference ACI 349-13.

The loads generated from a design-basis accident in load combinations (9-9) and (NB2-9), Y_r, Y_j, and Y_m, are concentrated loads and their effect is mainly local. Pipe reactions during normal operation and abnormal conditions, R_o, and R_a, are also local effects. Thus, these loads are excluded from the loads that have a major effect in the in-plane direction of walls and slabs.

The maximum differential pressure load, P_a, fluid pressure load, F, soil pressure load, H, and thermal loads during normal and abnormal conditions, T_o, and T_a, are expected to have a major effect on the out-of-plane flexure of walls and slabs but only a minor effect in their in-plane direction. Therefore, these loads are omitted. The resulting load combination to evaluate in-plane cracking for walls and slabs in the seismic load- resisting system is shown below:

$$U = D + 0.8L + E_{ss} \qquad \text{Equation 4-1}$$

The member forces due to E_{ss} are obtained from the TRB Seismic model and consist of force time histories. The member forces due to $D + 0.8L$ are obtained from the TRB Static model and added to the E_{ss} forces at each time step. Thus, the resultant member forces from Equation 4-1 are also time histories. The maximum force along the time history is used to determine the cracking state of the member.

The load combination in Equation 4-1 is only used to evaluate the state of cracking in walls and slabs of the seismic force resisting system. The full load combinations in ACI 349 or AISC N690-18 are used to obtain the member forces for design.

4.1.2 Effective Stiffness and Damping Ratio Assignment

As explained above, in-plane cracking is evaluated in the lateral-load-resisting members. The RL is evaluated for ISRS generation only. Damping values corresponding to RL 2 are used for member design.

As shown at the top of Figure 4-1, the TRB Seismic model is initially set to {{ }}2(a),(c) damping and uncracked in-plane stiffness for members. The TRB Static model is also assigned uncracked in-plane stiffness for members. At this point, members in the seismic load resisting system are assigned {{ }}2(a),(c).

The process to assign effective stiffness and damping ratios is shown in green in Figure 4-1 and is described as follows:

1. Using Equation 4-1, obtain the in-plane shear stress, v , and the in-plane bending stress, f_b , at critical sections in the main lateral force resisting system walls and slabs. Critical sections extend the total member length or width for both SC walls and RC members. For soil types 7 and 11, a representative in-column input motion is considered as the SSE in load combination 4-1 (e.g., Capitola). For soil type 9, the only in-column input motion is Lucerne.
2. Compare the maximum stresses v and f_b obtained for each member with the cracking limits specified in Section 4.1. If the maximum stresses in the member exceed any of these limits, assign the corresponding effective (cracked) stiffness, for the same members change the RL to {{ }}2(a),(c) for ISRS generation.
3. For ISRS generation, using the calculated RL, assign a damping ratio to the members in the TRB Seismic model based on Table 4-1. Damping values corresponding to {{ }}2(a),(c) are assigned to the TRB Seismic model for generation of member forces for design.

The same in-plane stiffness is used for the lateral-load-resisting members in the TRB Static model, when this model is used to obtain member forces due to the non-seismic loads in load combinations (9-6) and (9-9) in ACI 349 for concrete structures, and load combinations (NB2-6) and (NB2-9) in AISC N690-18 for steel structures. For the remaining load combinations not involving seismic loads, uncracked, in-plane stiffness is used for members in the TRB Static model.

For SC walls, for load combinations considering accident thermal loads, T_a (i.e., load combinations (NB2-8) and (NB2-9) in AISC N690-18), the out-of-plane flexural stiffness is further reduced as required by AISC N690-18.

4.2 Force Calculation and Member Design

To start the design process, the structural members have dimensions and reinforcement corresponding to the minimum requirements of ACI 349 or AISC N690-18.

4.2.1 Member Forces due to Non-Seismic Loads

The member forces due to load combinations not involving seismic loads are obtained from the TRB Static model with uncracked, in-plane stiffness for members. For load combinations involving seismic loads, the member forces are obtained from the TRB Static model with in-plane stiffness of the lateral-load-resisting members matching the in-plane stiffness in the TRB Seismic model (Figure 4-1).

For concrete members, ACI 349 requires that the structural effects of differential settlement be included as part of the dead load, *D*, in load combinations (9-4) through (9-9). For SC walls, AISC N690-18 requires these effects be included with the soil pressure load, *H*. As specified previously, the TRB static model includes the half-space corresponding to soil type 11 with stiffness reduced by half. Therefore, the structural effects of differential settlement are included in the calculation of the member forces.

4.2.2 Member Force due to Seismic Loads

Because the TRB Seismic model is updated with the stiffness and damping obtained in Section 4.1.2, a new harmonic analysis is performed for the required frequencies (Figure 4-1).

After the harmonic analysis has been rerun with the updated stiffness and damping, the member forces corresponding to E_{ss} are obtained.

4.2.3 Design Process

The design process can be performed independently for each of the three representative buildings (RXB, CRB, and RWB). The process is shown in red in Figure 4-1 and is described as follows:

1. For load combinations involving seismic loads, combine member forces from non-seismic and seismic loads at each time step. The in-plane stiffness of the lateral load resisting members in the TRB Seismic and TRB Static model are matched. For load combinations not involving seismic loads, the member forces are obtained from the TRB Static model with uncracked in-plane stiffness for members.
2. For each load combination and each action (i.e., in-plane shear), obtain DCR. For load combinations involving seismic loads, the DCR ratio is the maximum over the

time history. The DCR is less than 1.0 in these cases. Add reinforcement if needed.

3. For load combinations involving seismic loads, average DCR due to CSDRS-compatible input motions.
4. Envelop reinforcement due to these load combinations and obtain controlling DCR for each member.

Repeat the process for each soil type. Final reinforcement is the envelope of the required reinforcement for assumed soil types; that is, for soil type 11 and CSDRS-compatible ground motions; for soil type 7 and CSDRS-compatible ground motions; and for soil type 9 and CSDRS-HF-compatible ground motion.

4.3 In-Structure Response Spectra Calculation

Because the TRB Seismic model is updated with the stiffness and damping obtained in Section 4.1.2, a new harmonic analysis is performed for the required frequencies (Figure 4-1). Using the updated TRB Seismic model, ISRS are obtained as shown in blue in Figure 4-1 and as described below:

1. For each in-layer motion, obtain ISRS at the required locations using the harmonic results from the updated TRB seismic model. The ISRS are obtained after algebraic summation of the acceleration time history responses in one particular direction, due to input motion in the X, Y, and Z directions. ISRS are obtained for the following percentages of the critical damping: 2 percent, 3 percent, 4 percent, 5 percent, 7 percent, and 10 percent.
2. Average ISRS due to the CSDRS-compatible input motions. Because there is only one set of CSDRS-HF-compatible input motion, no averaging is needed for this set.
3. Envelop the ISRS at the nodes in each floor region to obtain the ISRS for that floor region.
4. For ISRS obtained in step 3, perform peak broadening considering +/-15 percent peak broadening, in agreement with Regulatory Guide 1.122 (Reference 10.1.7).

This process is repeated for each soil type. The final ISRS is the envelope of the ISRS for the soil types (that is, for soil types 7 & 11 and CSDRS-compatible ground motions, and for soil type 9 and certified seismic design response spectra - high frequency (CSDRS-HF)-compatible ground motion).

5.0 Effective Stiffness for Seismic Category I and Category II Structures Analysis

5.1 Purpose

The purpose of this section is to describe modeling approaches to represent effective stiffness for RC wall, RC slab members and for SC walls.

5.2 Applicability

This methodology applies to modeling the Seismic Category I and II structures, namely, RXB, the CRB, and the RWB of a representative SMR. These structures are primarily comprised of thick RC basemat, walls, and slabs and can include SC walls. Therefore, this section presents methodologies for the inclusion of effective stiffness in building models comprised of either RC members or SC walls.

5.3 Effective Stiffness for Concrete Members

In accordance with ASCE 4-16, in lieu of detailed stiffness calculations, an effective stiffness of RC walls and slabs in safety-related buildings can be modeled using the factors shown in Reference 10.1.2. Mat foundations are not included in the ASCE 4-16 stiffness requirements. Per Reference 10.1.19, finite element analysis of mat foundations typically assumes gross section concrete stiffness. Therefore, the representative building's basemats are considered uncracked.

Table 3-1 of ASCE 43-19 (Reference 10.1.1) shows the limits to determine whether walls and slabs (or diaphragms) are considered uncracked or cracked. Considering the flexural rigidity, walls and slabs are considered cracked if the following condition holds true; otherwise, they are considered uncracked:

$$f_b > f_{cr}$$

where f_b is the bending stress and f_{cr} is the cracking stress obtained from ACI 318-14 (Reference 10.1.9) Equation 19.2.3.1, shown below

$$f_{cr} = 7.5\sqrt{f_c'}$$

for normal weight concrete, with f_c' as the concrete compressive strength in psi.

Considering the shear rigidity, walls and slabs are considered cracked if the following condition holds true; otherwise, they are considered uncracked:

$$V > V_c$$

where, V is the wall shear, and, V_c is the nominal concrete shear capacity obtained from ASCE 4-16, as shown below:

$$V_c = A_w 3\sqrt{f_c'}$$

where A_w is the web area.

For out-of-plane responses of walls and slabs, the out-of-plane flexural stiffness is reduced by 50 percent, while the shear rigidity is not reduced. For design-basis shaking, the out-of-plane flexural moments typically result in flexural stress that exceeds the cracking strength, and, therefore, the reduced out-of-plane flexural stiffness is recommended for analysis of design-basis shaking. In contrast, shear stresses are typically relatively low; therefore, walls may not be cracked by shear, and consideration of the full uncracked condition is appropriate. For in-plane bending and shear, the cracked flexural and shear stiffness are both reduced by 50 percent, while the axial stiffness is not reduced.

5.4 Effective Stiffness for Steel-Plate Composite Walls

The stiffness requirements for SC walls are specified in Section N9.2.2 of AISC N690-18 (Reference 10.1.3). Effective stiffness is calculated for operating and accident thermal conditions.

The out-of-plane flexural stiffness is based on the stiffness of the cracked transformed section, including the faceplates and the cracked concrete infill. The effective flexural stiffness, per unit width of wall, is calculated with AISC N690-18 Equation A-N9-8.

The in-plane shear stiffness is evaluated separately for the operating and accident conditions.

For operating conditions, the effective in-plane shear stiffness, per unit width of wall, depends on the ratio of the average in-plane shear required strength, S_{rxy}, to the concrete cracking threshold, S_{cr}. AISC N690-18 specifies a trilinear relationship in which:

$$\text{if } S_{rxy} \le S_{cr}$$

the effective shear stiffness corresponds to the uncracked shear stiffness, and is calculated with AISC N690-18 Equation A-N9-9.

$$\text{if } S_{rxy} > 2S_{cr}$$

the effective shear stiffness corresponds to the cracked shear stiffness, and is calculated with AISC N690-18 Equation A-N9-14.

$$\text{if } S_{cr} \le S_{rxy} \le 2S_{cr}$$

the effective shear stiffness is linearly interpolated between the cracked and uncracked shear stiffness, using AISC N690-18 Equation A-N9-11.

The concrete cracking threshold is obtained as

$$s_{cr} = \frac{0.063\sqrt{f_c'}}{G_c} GA_{uncr}$$

where f_c' is the concrete compressive strength in ksi, G_c is the concrete shear modulus, and GA_{uncr} is the gross (uncracked) in-plane shear stiffness of the SC section.

For accident thermal conditions, the in-plane shear stiffness, per unit width of wall, is taken as that for the cracked case.

In AISC N690-18, recommendations are not included for the effective in-plane flexural stiffness or the effective in-plane axial stiffness. For walls modeled with shell or solid-shell finite elements, the in-plane flexural stiffness results from the integration of the axial stiffness of the finite elements comprising the wall, along its cross section. Therefore, a reduction in the wall in-plane axial stiffness requires a reduction in the wall in-plane flexural stiffness.

The effective in-plane axial stiffness captures both the reduced stiffness of the wall in tension and the gross stiffness of the wall in compression. Thus, an effective axial stiffness, per unit width, is calculated as the average of the cracked and uncracked stiffness values, as shown below.

$$EA_{eff} = E_s A_s + \frac{E_c A_c}{2} \qquad \text{Equation 5-1}$$

where E_s and E_c are the steel and concrete Young's modulus, respectively, and A_s and A_c are the steel and concrete sectional area per unit width.

This effective in-plane axial stiffness, and the resulting in-plane effective flexural stiffness, are used for the cracked concrete case.

A summary of the stiffness requirements for SC walls is shown in Table 5-1 below.

Table 5-1 Effective stiffness for steel-plate composite walls

Structural Action	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Out-of-plane	EI_{eff}[1]	GA_c	EA_{gross}
In-plane (uncracked)	EI_{gross}	GA_{uncr}[2]	EA_{gross}
In-plane (cracked)	EI_{in_plane}[3]	GA_{cr}[4]	EA_{eff}[5]

[1] Obtained with Equation A-N9-8 of AISC N690-18

[2] Obtained with Equation A-N9-9 of AISC N690-18 (same as gross)

[3] As a result of effective axial stiffness

[4] Obtained with Equation A-N9-12 of AISC N690-18

[5] Obtained with Equation 5-1

According to Section N9.2.3 of AISC N690-18, an elastic finite element model of the composite SC section can be developed using a single material with the following properties.

- The model Poisson's ratio is taken as that of the concrete.
- The model section thickness and the material elastic modulus are established through calibration to match the effective stiffness values for analysis, EI_{eff} and GA_{eff}.
- The model material density is obtained after calibration to match the mass of the SC section.

5.5 Methodology

The preceding discussions demonstrate the effective stiffness requirements for RC members and SC walls are similar; thus, the methodology presented herein applies to both.

Traditionally, seismic analysis of nuclear power plant facilities has been performed using finite element models with isotropic material. To apply the effective stiffness requirements of ASCE 4-16 or AISC N690-18, a compromise between the stiffness values for the different structural actions (e.g., in-plane and out-of-plane flexure, shear and axial actions) is necessary.

For the seismic analysis of the representative SMR Seismic Category I and II facilities, finite element models are built using ANSYS (Reference 10.1.8), which has different types of material models. The methodology presented herein makes use of the ANSYS orthotropic material model. Thus, the Young's modulus and shear modulus can be independently defined for each of the three finite element orthogonal local axes (Figure 5-1).

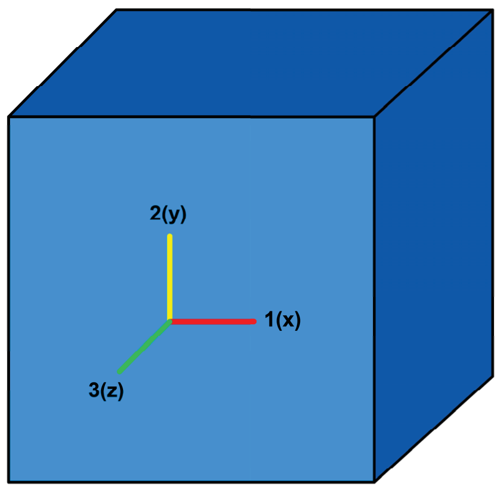
5.5.1 Effective Stiffness in the Finite Element Model

The finite element models of the buildings contain traditional shell and three-dimensional, solid-shell type elements. The ANSYS solid-shell elements (SOLSH190)

(Reference 10.1.8) behave similarly to traditional shell elements with the difference that SOLSH190 is compatible with general three-dimensional constitutive relations and can be connected directly with other continuum elements. The plane of the element is defined by the use of a normal vector (i.e., the normal vector is perpendicular to the plane of the element). For walls, the element plane is vertical; for slabs, the element plane is horizontal.

The local axes of the solid-shell elements is shown in Figure 5-1. Local axes 1(x) and 2(y) are in the element plane (e.g., vertical plane for walls or horizontal plane in slabs), and local axis 3(z) is perpendicular to that plane. These local axes are identical for shell elements.

Figure 5-1 Solid shell element local axes in ANSYS



The effective stiffness values are implemented in the finite element model using a similar approach developed for SC walls (Section 5.4). That is, the model section thickness and the orthotropic material properties are obtained through calibration to match the effective stiffness values specified by ASCE 4-16 or AISC N690-18.

Thus, the model thickness, t_m, and the model in-plane Young's modulus, E_m, are obtained by equating the out-of-plane flexural stiffness and in-plane axial stiffness to the corresponding effective stiffnesses per unit width of wall or slab. The following equations are obtained:

$$\frac{E_m t_m^3}{12} = EI_{eff} \qquad \text{Equation 5-2}$$

$$E_m t_m = EA_{gross} \text{ or } EA_{eff} \qquad \text{Equation 5-3}$$

where EI_{eff} is the effective out-of-plane flexural stiffness, and EA_{eff} is the effective axial stiffness.

Solving Equation 5-2 and Equation 5-3, the model section thickness and in-plane Young's modulus are obtained with:

$$t_m = \sqrt{\frac{12EI_{eff}}{EA_{gross} \text{ or } EA_{eff}}} \qquad \text{Equation 5-4}$$

$$E_m = \frac{12EI_{eff}}{t_m^3} \qquad \text{Equation 5-5}$$

The model in-plane shear modulus, G_m, is obtained with the following equation, after the model thickness, t_m, has been calculated:

$$G_m = \frac{GA_{eff}}{t_m} \qquad \text{Equation 5-6}$$

where GA_{eff} is the effective (gross or cracked) in-plane shear stiffness per unit width of wall or slab.

The model unit weight for RC members is calculated with the following equation, after the model thickness, t_m, has been calculated:

$$\gamma_m = \frac{w_c t_o}{t_m} \qquad \text{Equation 5-7}$$

where w_c is the concrete unit weight, and t_o is the original RC member thickness.

The model unit weight for SC walls is calculated with the following equation, after the model thickness, t_m, has been calculated:

$$\gamma_m = \frac{\gamma_s 2t_p + w_c t_c}{t_m} \qquad \text{Equation 5-8}$$

where γ_s is the steel unit weight, t_p is the thickness of the faceplates, and, t_c is the thickness of the concrete infill.

Additionally, for both RC and SC members:

- the model shear stiffness in the out-of-plane direction per unit width of plate, G_{m-op}, is taken as that of the concrete, and
- the model through-thickness axial stiffness per unit area, E_{m-z}, is taken as that of the concrete

Thus;

$$G_{m-op} = \frac{G_c A_c}{t_m} \qquad \text{Equation 5-9}$$

$$E_{m-z} = E_c \qquad \text{Equation 5-10}$$

For the cracked case, the resulting model thickness and in-plane Young's modulus, t_m and E_m, respectively, reduce the in-plane flexural stiffness in the same proportion as the in-plane axial stiffness. Examples for the calculation of effective stiffness for both RC members and SC wall are presented in Section 5.6.

5.5.2 Implementation of Effective Stiffness Values for Solid-Shell Elements

Two alternate methods are used for implementing the effective stiffness values using ANSYS SOLSH190 elements. Both methods use three-layered SOLSH190 elements. In both methods the equivalent material properties are orthotropic. The methods differ in layer thickness and material properties, however both result in section stiffness equal to the effective section stiffness defined by AISC N690-18, Section N9.2.2.

Method 1 is described in Section 5.5.2.1. A single material is used for the middle layer in Method 1. The outer two layers are dummy layers, having zero density, and insignificant value of Young’s modulus and shear modulus, e.g. Young’s Modulus, E ≤ 10 kip/ft^2, Poisson’s Ratio V=0.17, and shear modulus G=E/2(1+V). In Method 1, the middle layer has an equivalent model thickness and effective elastic model properties as defined by ASCE 4-16 (Reference 10.1.2) and AISC N690-18. Method 1 used a single orthotropic material as in AISC N690-18, Section N9.2.3.

Method 1 is applicable for both RC and SC walls. The finite element mesh has node spacing equal to the actual RC or SC wall thickness. In some analysis cases when using SC walls, the model thickness is greater than the actual SC wall and the finite mesh must be remeshed with a greater node spacing. The need to remesh is avoided when Method 2 can be used.

Method 2 is described in Section 5.5.2.2. In Method 2, the middle layer and outer layers have elastic properties and thicknesses defined to be equivalent to the effective section stiffness as presented in ASCE 4-16 and AISC N690-18. The thicknesses of outer layers are equal or greater than the actual faceplate thickness. The model uses different material properties for the inner and outer layers to match the effective stiffness defined by AISC N690-18, Section N9.2.2.

Method 2 is applicable for SC walls only. The finite element mesh has node spacing equal to the actual SC wall thickness.

5.5.2.1 Method 1 for Implementing Effective Stiffness Using SOLSH190 Elements

The model section thickness and orthotropic material properties obtained as specified in Section 5.5.1, are implemented in ANSYS shell finite elements. For solid-shell finite elements, however, a new model is created for each, resulting model thickness t_m.

For the cases in which the model thickness, t_m, is smaller than the actual solid-shell element thickness, a work-around is to change the single-layered solid-shell elements (by default) to multi-layered elements. This is done by associating the solid-shell element with a shell section using APDL command SECTYPE. Three layers are then assigned to the section as shown in Figure 5-2. The middle layer has the resulting model thickness, t_m, and the orthotropic material properties calculated for cracked or uncracked cases. The outer two layers are dummy layers, and have zero density and insignificant value of Young's modulus and shear modulus, e.g. Young's Modulus, E ≤ 10 kip/ft^2, Poisson's Ratio V=0.17, and shear modulus G=E/2(1+V), so their contribution to the total element stiffness is negligible.

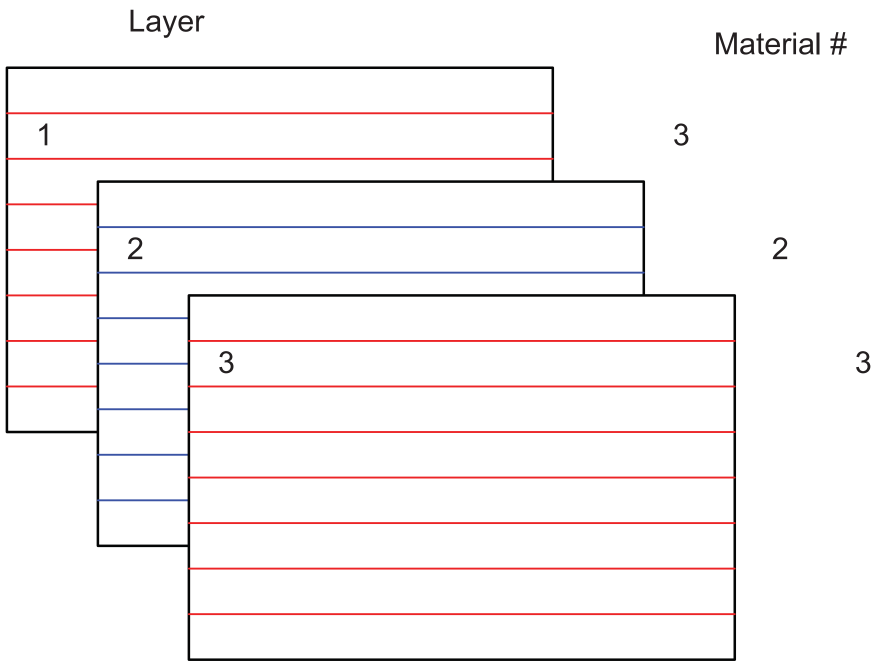
The thickness of the outer layers (layers 1 and 3 in Figure 5-2) is calculated as

$$t_{1,3} = \frac{t - t_m}{2}$$

where t is the original element thickness.

The ANSYS model obtains the actual layer thicknesses used for element calculations by scaling the input layer thickness so that they are consistent with the thickness between the nodes. Thus, to use multi-layered elements, the resulting model thickness, t_m, has to be smaller than the actual element thickness.

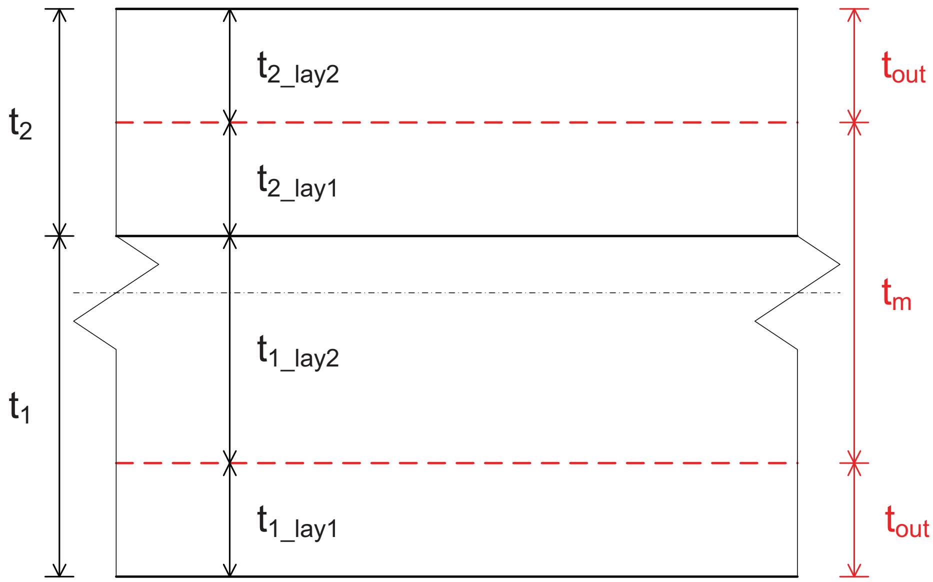
Figure 5-2 Three-layered solid-shell element



For RC or SC members modeled with more than one row of solid-shell elements through the thickness, an equivalent middle layer with thickness, t_m, and two dummy outer layers, t_{out}, are defined as follows. For a member modeled with two rows of solid-shell elements, as shown in Figure 5-3, the middle layer is centered with respect to the total member thickness. The thickness of the dummy outer layers is then obtained as follows:

$$t_{out} = \frac{(t_1 + t_2 - t_m)}{2}$$

Figure 5-3 Member modeled with two rows of solid-shell elements through the thickness



On Figure 5-3, the thicknesses t_1 and t_2 refer to thicknesses of the two rows of finite elements and not material layer numbers. Each row of elements has two material layers. Material layers within the two rows of finite elements are labeled "$t_{1\text{-lay1}}$", "$t_{1\text{-lay2}}$", "$t_{2\text{-lay1}}$"and "$t_{2\text{-lay2}}$". To assign the middle and dummy layers to the two rows of finite elements, two section layers are defined through the thickness of each solid-shell element. The section layer thicknesses are obtained as follows:

$$t_{1-lay1} = t_{out}$$
$$t_{1-lay2} = t_1 - t_{out}$$
$$t_{2-lay1} = t_2 - t_{out}$$
$$t_{2-lay2} = t_{out}$$

A similar approach can be used for members modeled with more than two rows of finite elements through the thickness.

Referring to Figure 5-1, model material properties are assigned to the orthotropic material as follows:

$$E_x = E_y = E_m$$
$$E_z = E_{m-z}$$
$$G_{xy} = G_m$$
$$G_{yz} = G_{xz} = G_{m-op}$$

$$v_{xy} = v_{xz} = v_{yz} = v_c$$

where v_c is the concrete Poisson's ratio.

5.5.2.2 Method 2 for Implementing Effective Stiffness Using SOLSH190 Elements

Method 2 is an alternate method for representing SC walls using ANSYS SOLSH190 elements. Method 2 uses two materials within a three-layered SOLSH190 element that has a total thickness equal to the actual wall thickness, as shown in Figure 5-4. The outer layers are chosen to have a thickness equal to or greater than the steel faceplates, while the middle layer has the same or less than the thickness as the SC wall concrete thickness. Different material properties are used for the outer and middle layers. In-plane values of Young's modulus and shear modulus are assigned to produce a section stiffness equivalent to effective stiffness of the SC wall (EI_{eff}, EA_{eff} and GA_{eff}) defined in Section 5.4 and summarized in Table 5-2. The effective flexural stiffness per unit width, EI_{eff}, is defined by Equation A-N9-8 in AISC N690-18, Section N9.2.2(a). The effective in-plane shear stiffness per unit width, GA_{eff}, is defined in Sections N9.2.2(b) and N9.2.2(c). For cracked conditions, the effective in-plane axial stiffness per unit width, EA_{eff}, is defined by Equation 5-1 of this standard. For uncracked conditions the in-plane axial stiffness per unit width is based on the gross cross section.

Figure 5-4 Two-material SOLSH190 element for method 2

{{

}}2(a),(c)

Table 5-2 Effective stiffness values used for cracked and uncracked conditions

Effective Stiffness (Note 1)	Uncracked condition		Cracked condition	
	Stiffness	**Definition**	**Stiffness**	**Definition**
EI_{eff}	EI_{eff}	Equation A-N9-8 (Reference 10.1.3)	EI_{eff}	Equation A-N9-8 (Reference 10.1.3)
EA_{eff}	EA_{gross}	$E_s A_s + E_c A_c$	EA_{eff}	$EA_{eff} = E_s A_s + \frac{E_c A_c}{2}$ Equation 5-1
GA_{eff}	GA_{uncr}	Equation A-N9-9 (Reference 10.1.3)	GA_{cr}	Equation A-N9-12 (Reference 10.1.3)

Note 1: The stiffnesses EI_{eff}, EA_{eff} and GA_{eff} are used in Equation 5-11, Equation 5-12, and Equation 5-27.

5.5.2.2.1 Equivalent Young's Modulus of Inner and Outer Layers

The model in-plane Young's modulus of the outer layers, E_{ms}, and the inner layers E_{mc}, are obtained by equating the model out-of-plane flexural stiffness and model in-plane axial stiffness to the corresponding effective stiffness (EI_{eff} and EA_{eff}) per unit width of wall or slab. The following equations express this relationship:

$$E_{ms} I_s + E_{mc} I_c = EI_{eff} \quad \text{Equation 5-11}$$

$$E_{ms} A_s + E_{mc} A_c = EA_{eff} \quad \text{Equation 5-12}$$

To represent different assumed cracking conditions, in Equation 5-11 and Equation 5-12, EI_{eff} and EA_{eff} are assigned values defined by Table 5-2.

In Equation 5-11 and Equation 5-12, the values of A_s, I_s, A_c, I_c are initially calculated by assuming the thickness of the outer layers, t_s, to be the same as the actual thickness of the steel faceplates. In some cases, (for example, uncracked conditions with thin faceplates) the solution of Equation 5-11 and Equation 5-12 for E_{ms} and E_{mc} results in a negative value for E_{ms}. In these cases either Method 2 can be used by increasing the assumed thickness of the outer layers as explained later, or alternatively, for these cases Method 1 can be used.

Solving for E_{ms} and E_{mc} using Cramer's rule:

$$E_{ms} = \frac{EI_{eff} A_c - EA_{eff} I_c}{\Delta} \quad \text{Equation 5-13}$$

$$E_{mc} = \frac{EA_{eff} I_s - EI_{eff} A_s}{\Delta} \qquad \text{Equation 5-14}$$

where

$$\Delta = I_s A_c - A_s I_c \qquad \text{Equation 5-15}$$

The value of Δ defined by Equation 5-15 is always positive if

$$I_s A_c > A_s I_c \qquad \text{Equation 5-16}$$

$$\frac{I_s}{A_s} > \frac{I_c}{A_c} \qquad \text{Equation 5-17}$$

$$r_s^2 > r_c^2 \qquad \text{Equation 5-18}$$

Where r_s and r_c are the radii of gyration of the faceplate and concrete respectively. Because the radius of gyration of the outer layers is larger than that of the middle layer, it is true that the denominator of Equation 5-13 and of Equation 5-14 is always positive.

Examining the numerator of Equation 5-13, the modulus of the outer layers E_{ms} is positive if:

$$\frac{EA_{eff}}{EI_{eff}} < \frac{A_c}{I_c} \qquad \text{Equation 5-19}$$

The minimum value of EI_{eff} occurs under accident conditions (Large ΔT_{savg}) and is equal to $E_s I_s$ (AISC N690-18 Equation A-N9-8), while the largest possible value of EA_{eff} is $E_s A_s + E_c A_c$ (for uncracked concrete in axial compression). Inserting these into Equation 5-19 the condition for the numerator of Equation 5-13 to be positive is:

$$\frac{E_s A_s + E_c A_c}{E_s I_s} < \frac{A_c}{I_c} \qquad \text{Equation 5-20}$$

Using the modular ratio $n = E_s/E_c$ gives

$$\frac{2nt_s + t_c}{\frac{nt_s(t_c + t_s)^2}{2}} < \frac{t_c}{\frac{t_c^3}{12}} \qquad \text{Equation 5-21}$$

$$\frac{t_c^2\ (2nt_s + t_c)}{6nt_s(t_c + t_s)^2} < 1 \qquad \text{Equation 5-22}$$

The modulus E_{ms} is generally positive except for relatively thin face plates combined with low effective flexural stiffness and high axial stiffness as in uncracked conditions; E_{ms} is generally positive for cracked conditions. Also, E_{ms} is generally positive for uncracked conditions with relatively thick faceplates and flexural stiffness is based operating conditions (ΔT_{savg}=0).

For cases where the value of E_{ms} calculated from Equation 5-13 is negative, there are two options:

1. The thickness of the outer layers is increased by a factor α >1 while maintaining the same total thickness, t_{sc}. Modified outer layer thicknesses $t'_p = \alpha t_p$ and middle layer thickness $t'_c = t_{sc} - 2\alpha t_p$ replace the values of t_p and t_c used to define the terms (A_s, I_s, A_c, I_c) in Equation 5-13, Equation 5-14, and Equation 5-15. The value of α is arbitrary and can be selected by trial and error to result in a positive value of E_{ms}.

   The out-of-plane moduli (EZ, GXZ, GYZ) must be scaled to the new layer thicknesses. The in-plane model shear modulus, GXY, calculated in Section 5.5.2.2.2, must also be adjusted using the modified layer thicknesses.

   The example problem in Section 5.6.3 uses a value of α = 2 to illustrate this option for the case for an uncracked 4-ft wall with three-quarter-inch faceplates.
2. Method 1 can be used. In these cases the effective thickness of the middle layer is less than t_{sc} and thickness of the two dummy layers is greater than zero.

Examining the numerator of Equation 5-14, the modulus of the middle layer E_{mc} is positive if

$$\frac{EI_{eff}}{EA_{eff}} < \frac{I_s}{A_s} \qquad \text{Equation 5-23}$$

The minimum value of EI_{eff} is equal to E_sI_s (AISC N690-18, Equation A-N9-8), while the largest possible value of EA_{eff} is $E_sA_s + E_cA_c$ (for uncracked concrete in axial compression). Inserting these into Equation 5-18, the condition for the numerator of Equation 5-14 to be positive is

$$\frac{E_s I_s}{E_s A_s + E_c A_c} = \frac{I_s}{A_s + \frac{E_c}{E_s} A_c} < \frac{I_s}{A_\mathrm{s}} \qquad \text{Equation 5-24}$$

Because this is always true, the modulus of the middle layer, E_{mc} is always positive.

5.5.2.2.2 Equivalent Shear of Inner and Outer Layers

The model in-plane shear modulus of the outer layers, G_{ms}, and model in-plane shear modulus of the inner layers, G_{mc}, are obtained by equating the model shear stiffness to the corresponding effective stiffness (GA_{eff}) per unit width of wall or slab. To do this, the concrete and steel shear moduli are scaled by a factor f_g. The following equations express this relationship:

$$G_{ms} = f_g G_s \qquad \text{Equation 5-25}$$

$$G_{mc} = f_g G_c \qquad \text{Equation 5-26}$$

$$f_g G_s A_s + f_g G_c A_c = GA_{eff} \qquad \text{Equation 5-27}$$

$$f_g = \frac{GA_{eff}}{G_s A_s + G_c A_c} \qquad \text{Equation 5-28}$$

To represent different assumed cracking conditions, in Equation 5-27 and Equation 5-28, GA_{eff} is assigned the value defined by Table 5-2.

5.5.2.2.3 Orthotropic Material Properties for Method 2

For the orthotropic material (Reference 10.1.8):

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{xz} \end{Bmatrix} = \begin{bmatrix} 1/E_x & -\nu_{xy}/E_x & -\nu_{xz}/E_x & 0 & 0 & 0 \\ -\nu_{yx}/E_y & 1/E_y & -\nu_{yz}/E_y & 0 & 0 & 0 \\ -\nu_{zx}/E_z & -\nu_{zy}/E_z & 1/E_z & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/G_{xy} & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/G_{yz} & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/G_{xz} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{xz} \end{Bmatrix} \qquad \text{Equation 5-29}$$

There are nine independent elastic constants E_x, E_y, E_z, G_{xy}, G_{yz}, G_{xz} and either ν_{xy}, ν_{yz}, ν_{xz} or ν_{yx}, ν_{zy}, ν_{zx}. The first six constants are input to ANSYS as the material properties EX, EY, EZ, GXY, GYZ, and GXZ.

The three values ν_{xy}, ν_{yz}, and ν_{xz} are called major values of Poisson's ratio and are input into ANSYS as the material properties PRXY, PRYZ, and PRXZ. The three values ν_{yx}, ν_{zy}, and ν_{zx} are called minor values of Poisson's ratio and are input into ANSYS as the material properties NUXY, NUYZ, and NUXZ. Input to ANSYS can consist of either major or minor Poisson's ratio. If PRXY, PRYZ, and PRXZ are input, the values of NUXY, NUYZ, and NUXZ are calculated (see Equation 5-33, Equation 5-34, and Equation 5-35).

Noting the symmetry of the elastic flexibility matrix on the right side of Equation 5-29, the two sets of Poisson's ratios are related by

$$\upsilon_{yx} = \frac{E_y}{E_x}\upsilon_{xy} \qquad \text{Equation 5-30}$$

$$\upsilon_{zy} = \frac{E_z}{E_y}\upsilon_{yz} \qquad \text{Equation 5-31}$$

$$\upsilon_{zx} = \frac{E_z}{E_x}\upsilon_{xz} \qquad \text{Equation 5-32}$$

or in ANSYS notation

$$\mathrm{NUXY} = \frac{\mathrm{EY}}{\mathrm{EX}}\mathrm{PRXY} \qquad \text{Equation 5-33}$$

$$\mathrm{NUYZ} = \frac{\mathrm{EZ}}{\mathrm{EY}}\mathrm{PRYZ} \qquad \text{Equation 5-34}$$

$$\mathrm{NUXZ} = \frac{\mathrm{EZ}}{\mathrm{EX}}\mathrm{PRXZ} \qquad \text{Equation 5-35}$$

The values NUXY, NUYZ, and NUXZ are named as minor values because when the axes are ordered EX>EY>EZ, then NUXY< PRXY, NUYZ< PRYZ, and NUXZ< PRXZ, i.e., if EX>EY>EZ the major values are greater than the minor values. For the materials defined for Method 2, EX=EY and EZ>EX. Therefore, for the current model, NUXY=PRXY, NUYZ>PRYZ, and NUXZ>PRXZ, (i.e., here the minor values NUYZ and NUXZ are greater than their corresponding major values in contrast to convention used to name them minor versus major).

The in-plane material properties are, E_x, E_y, and G_{xy} (EX, EY, and GXY) are equal to E_{ms}, E_{ms}, and G_{ms} for the outer layers and E_{mc}, E_{mc}, and G_{mc} for the middle layer. The value of Poisson's ratio for in-plane shear (ν_{xy} = ν_{yz} , PRXY=NUXY) is taken the value of Poisson's ratio for concrete ν_c = 0.17. The same Poisson's ratio, NUXY=0.17, is used for the entire section.

The remaining elastic constants [(EZ, GYZ, GXZ) and either (PRYZ, PRXZ) or (NUYZ, NUXZ)] are set to the values for a fully uncracked section. For the middle layer (EZ, GYZ, GXZ)= (E_c, G_c, G_c), while for the outer layers (EZ, GYZ, GXZ)= (E_s, G_s, G_s)

Because the code states that a single value (ν_c = 0.17) of Poisson's ratio is representative for entire section, a choice is necessary as to whether to assign the value 0.17 to either the major values (PRYZ and PRXZ) or the minor values (NUYZ and NUXZ) when they are not equal.

When material test data is provided, the convention is typically to provide major values that can be input to ANSYS as the major values of Poisson's ratio (PRYZ and PRXZ). However, for the current application, the choice is based on directions influenced by cracking and consideration that in the present application the stiffer direction is now the Z-direction. Based on the following discussion, the appropriate values to use are the minor values (NUYZ and NUXZ).

Equation 5-29 is first rewritten using the ANSYS notation for the material constants:

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{xz} \end{Bmatrix} = \begin{bmatrix} 1/EX & PRXY/EX & -PRXZ/EX & 0 & 0 & 0 \\ -NUXY/EY & 1/EY & -PRYZ/EY & 0 & 0 & 0 \\ -NUXZ/EZ & -NUYZ/EZ & 1/EZ & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/GXY & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/GYZ & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/GXZ \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{xz} \end{Bmatrix} \quad \text{Equation 5-36}$$

Intuitively, the equation for the strain in the Z-direction is not altered because of cracking, and the value of NUXZ and NUYZ is to be set to ν_c = 0.17. The reasoning for specifying NUXZ and NUYZ follows.

Consider whether to use the value NUXZ versus PRXZ. The same considerations apply for the YZ values. Consider the case of strain on the XZ plane with σ_y = 0 and ε_x = 0. Then:

$$\varepsilon_x = 0 = \frac{1}{\mathrm{EX}}(\sigma_x - \mathrm{PRXZ} \cdot \sigma_z) \quad \text{Equation 5-37}$$

$$\varepsilon_z = \frac{1}{\mathrm{EZ}}(\sigma_z - \mathrm{NUXZ} \cdot \sigma_x) \quad \text{Equation 5-38}$$

Solving Equation 5-37 for inserting into Equation 5-38 gives

$$\varepsilon_z = \frac{1}{\mathrm{EZ}}(1 - \mathrm{NUXZ} \cdot \mathrm{PRXZ})\sigma_z \qquad \text{Equation 5-39}$$

$$\sigma_z = \frac{\mathrm{EZ}\varepsilon_z}{1 - \mathrm{NUXZ} \cdot \mathrm{PRXZ}} \qquad \text{Equation 5-40}$$

The strain energy per unit volume is

$$\bar{V} = \frac{1}{2}\sigma_z\varepsilon_z = \frac{\frac{1}{2}\mathrm{EZ}\varepsilon_z^2}{1 - \mathrm{NUXZ} \cdot \mathrm{PRXZ}} \qquad \text{Equation 5-41}$$

For the fully uncracked case, EX = EZ and NUXZ·PRXZ = 0.17^2.

Now consider the cracked case where PRXZ is specified to be 0.17. Then $\mathrm{NUXZ} = \frac{\mathrm{EZ}}{\mathrm{EX}}0.17$. For the section with reduced in-plane stiffness EX and with EZ still set to the full uncracked stiffness, NUXZ > 0.17. Then NUXZ·PRXZ > 0.17^2. The denominators in the right side of Equation 5-40 and of Equation 5-41 are smaller for the cracked than uncracked case, making σ_z and $\bar{V}$ larger for the cracked than uncracked case. For a given strain ε_z, cracking produces larger strain energy $\bar{V}$ and greater stiffness σ_z / ε_z. This is unrealistic, because energy is released by cracking and stiffness reduced.

Now consider the cracked case where NUXZ is specified to be 0.17. Then $\mathrm{PRXZ} = \frac{\mathrm{EX}}{\mathrm{EZ}}0.17$. For the section with reduced in-plane stiffness EX and with EZ still set to the full uncracked stiffness, PRXZ < 0.17. Then NUXZ·PRXZ < 0.17^2. Thus, for a given strain ε_z, cracking produces smaller strain energy $\bar{V}$ and smaller stiffness σ_z / ε_z. This is realistic, because cracking releases strain energy and reduces stiffness. Similarly, it is concluded that the value NUYZ=0.17 be used.

The final result for the middle layer is

$$\mathrm{EX} = \mathrm{EY} = E_{mc} \qquad \text{Equation 5-42}$$

$$\mathrm{EZ} = E_c \qquad \text{Equation 5-43}$$

$$\mathrm{GXY} = G_{mc} \qquad \text{Equation 5-44}$$

$$\mathrm{GXZ} = \mathrm{GYZ} = G_c \quad \text{Equation 5-45}$$

$$\mathrm{NUXY} = \mathrm{NUXZ} = \mathrm{NUYZ} = \nu_c = 0.17 \quad \text{Equation 5-46}$$

The final result for the two outer layers is:

$$\mathrm{EX} = \mathrm{EY} = E_{ms} \quad \text{Equation 5-47}$$

$$\mathrm{EZ} = E_s \quad \text{Equation 5-48}$$

$$\mathrm{GXY} = G_{ms} \quad \text{Equation 5-49}$$

$$\mathrm{GXZ} = \mathrm{GYZ} = G_s \quad \text{Equation 5-50}$$

$$\mathrm{NUXY} = \mathrm{NUXZ} = \mathrm{NUYZ} = \nu_c = 0.17 \quad \text{Equation 5-51}$$

5.5.2.2.4 Summary of Steps to Apply Method 2 to Steel-Plate Composite Walls

Method 2 for SC walls is applied by following these steps:

1. Create finite element mesh of the SC walls using one three-layered SOLSH190 element through the wall thickness.
2. Generally, one element through the thickness is preferred. In cases where restrictions on meshing or other considerations require the use of more than one element through the thickness, the three layers are placed within the through thickness elements.
3. Define the elastic material properties for the concrete (E_c, G_c, and ν_c) and steel faceplates (E_s, G_s).
4. Define effective flexural stiffness per unit width (EI_{eff}), effective in-plane shear stiffness per unit width (GA_{eff}), and effective in-plane axial stiffness per unit width (EA_{eff}), as described in Table 5-2.
5. Assign the thickness of the middle layer to be the same as the thickness of the concrete and assign thickness of the outer layers to be the same as the steel faceplates.

   Calculate Young's modulus for the outside layers, E_{ms}, using Equation 5-13. Calculate Young's modulus for the middle layer, E_{mc}, using Equation 5-14. Calculate shear modulus for the outside layers, G_{ms}, using Equation 5-25. Calculate shear modulus for the middle layer, G_{mc}, using Equation 5-26.

Note that E_{mc}, G_{mc}, and G_{ms} are always positive. The modulus E_{ms} is generally positive for cracked conditions. For uncracked conditions, E_{ms} can be negative for relatively thin face plates; in this case, Method 1 can be used. As a second alternative, Method 2 can be used if the thickness of the outer layers can be increased by a factor $\alpha > 1$ while maintaining the same total thickness, t_{sc}. The value of α is arbitrary and can be selected by trial and error to result in a positive value of E_{ms}. If the second option is used, new values of E_{ms}, E_{mc}, G_{ms}, and G_{mc} are calculated using the revised layer thicknesses.

6. Assign elastic material properties of the middle layer using Equation 5-42 through Equation 5-46. Assign elastic material properties of the two outer layers using Equation 5-47 through Equation 5-51.
7. The material density assigned to the layer materials is determined to match the total mass of the model to the mass of the SC wall.

5.5.2.3 Implementing Effective Stiffness using SHELL181 Elements

SHELL181 is suitable for analyzing thin to moderately-thick shell structures. It has 24 degrees-of-freedom (DOF), i.e., four nodes with six DOF at each node, translations in the x, y, and z directions, and rotations about the x, y, and z axes.

The SOLSH190 element is useful for simulating a wide range of thickness (from thin to moderately thick). It also has 24 DOFs, i.e., eight nodes with three DOFs at each node, i.e., translations in the x, y, and z directions.

Therefore, both the SHELL181 and SOLID190 are suitable for analysis of SC and RC walls. The SOLSH190 element has the advantage that nodes are located on the surface of the wall or slab, which can make it easier to accurately represent the offset of interface with other elements or constraints among structural elements. For the RXB, examples include contact and constraints used for fluid-structure interaction with fluid elements. Offsets at connections between thick walls and slabs and other interfaces can be easier to represent using SOLSH190 elements. On the other hand, there can be cases where it is easier to mesh using SHELL181 elements.

As for the SOLSH190 element, the SHELL181 element can be defined as a single layer (single material) section or a layered composite section. Therefore, the single material model section calculated as in Method 1 or the multiple material layered model as in Method 2 can be used. In the case of Method 1, the model thickness would be input without the need for dummy layers, because only mid-plane nodes are used for the SHELL181.

Because both SOLSH190 and SHELL181 elements have 24 DOF, there is no significant difference in the overall number of DOF needed or the time for solving global equations. When the SHELL181 is used with a single layer versus three with the SOLSH190, there can be an advantage in time and storage for forming element stiffness and calculating and storing element results.

In Section 5.6.3, results are provided for test model using SHELL181 (Method 1) and compared to results using SOLSH190 (Methods 1 and 2).

5.6 Implementation Examples

5.6.1 Test Models using Method 1 for Implementing Effective Stiffness

Section 5.5.2.1 describes Method 1 for implementing the effective stiffness of RC and SC walls in finite element models. This section includes test models showing the implementation using Method 1.

The test models consist of a box-type structure modeled in ANSYS using solid-shell (SOLSH190) elements, as shown in Figure 5-5. The walls in this prototype structure have similar aspect ratios to the walls in the representative SMR Reactor Building. The short horizontal dimension (Y) is 60 ft, the long dimension (X) is 120 ft, and the height (Z) is 65 ft.

Two model sets are developed. In one set, the models have RC walls and slabs, and the thickness of members is 5 ft. In the other set, the walls consist of SC construction and slabs are RC. The SC wall thickness is reduced to 4 ft so the fundamental frequencies in the horizontal direction are similar to the ones of the model with RC members.

The model set with RC members consist of the following three models:

1. Model with gross (uncracked) stiffness values for members in x,y,z directions, referred to as “uncracked gross.” Using the gross stiffness value for effective out-of-plane flexural stiffness is conservative and bounding versus using the recommended stiffness from ASCE 4-16. This case bounds the stiffness of RC members in which the out-of-plane flexural stresses are low and shrinkage cracking is not significant. This limiting case is analyzed for comparison to other cases; however it is not recommended for the design of structures subject to the design basis seismic motion.
2. Model with gross (uncracked) stiffness values for members in x,y,z directions except for the out-of-plane flexure, in which the effective stiffness from ASCE 4-16 is used. This model is referred to as “uncracked EI_{cr}.”
3. Model with effectiveness values for the cracked case for members in x,y,z directions. This model is referred to as “cracked.”

The model set with SC walls consists of the following two models:

1. Model with gross (uncracked) stiffness values for members in x,y,z directions except for the out-of-plane flexure of the SC walls, in which the effective stiffness from Table 5-1 is used, and considering operational thermal condition. This model is referred to as “uncracked-SC.”

2. Model with effective stiffness values of SC walls from Table 5-1 for the cracked case, and operational thermal condition, for members in x,y,z directions. This model is referred to as “cracked-SC.”

The RC gross (uncracked) material properties are selected so that the main modal frequencies in the horizontal direction are similar to the main modal frequencies in the RXB. Thus, the unit weight is ω_c = 0.5kip/ft^3, the elasticity modulus is E_c = 4.104 × 10^{5}kip/ft^2, the shear modulus is G_c = 1.754 × 10^{5}kip/ft^2, and the material damping is chosen as {{ }}$^{2(a),(c)}$.

These properties are the same for the concrete portion of the uncracked-SC model. As explained above, the SC wall thickness is reduced to 4 ft so the main modal frequencies in the horizontal direction are similar to the RXB main frequencies. The SC walls have 3/4-in.-thick outer plates (faceplates). The steel plate building structure is shown in Figure 5-5.

Figure 5-5 Box-type building structure modeled with SOLSH190

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}}2(a),(c)

Model thickness and material properties for uncracked EI_{cr} model

The model thickness and material properties are obtained from Equation 5-4 through Equation 5-7, where EI_{eff} = 0.5EI_{gross}; EA_{gross} and GA_{eff} = GA_{gross}. The resulting values are:

$$t_m = \frac{5ft}{\sqrt{2}}$$

$$E_m = \sqrt{2}E_c$$

$$G_m = \sqrt{2}G_c$$

$$\Upsilon_m = \sqrt{2}w_c$$

Model thickness and material properties for cracked model

The model thickness and material properties are obtained from Equation 5-4 through Equation 5-7, using EI_{eff} = 0.5EI_{gross}, EA_{eff} = 0.5EA_{gross} and GA_{eff} = 0.5GA_{gross}. The resulting values are:

$$t_m = 5ft$$

$$E_m = 0.5E_c$$

$$G_m = 0.5G_c$$

$$\Upsilon_m = w_c$$

Model thickness and material properties for uncracked steel-plate composite model

The model thickness and material properties are obtained from Equation 5-4 through Equation 5-7, using EI_{eff}, EA_{gross} and $GA_{eff} = GA_{uncr}$, considering 3/4-in. faceplates and operating thermal condition. The resulting values are:

$$t_m = 3.788ft$$

$$E_m = 5.576 \times 10^5 kip/ft^2$$

$$G_m = 2.321 \times 10 \ \ kip/ft^2$$

$$\Upsilon_m = 0.528kip/ft^3$$

Model thickness and material properties for cracked steel-plate composite model

The model thickness and material properties are obtained from Equation 5-4 through Equation 5-7, using EI_{eff}, EA_{eff} and $GA_{eff} = GA_{cr}$, considering 3/4-in. faceplates and operating thermal condition. The resulting values are:

$$t_m = 4.797ft$$

$$E_m = 2.746 \times 10^5 kip/ft^2$$

$$G_m = 8.385 \times 10 \ \ kip/ft^2$$

$$\Upsilon_m = 0.417kip/ft^3$$

5.6.2 Modal Analysis of Test Models using Method 1 for Implementing Effective Stiffness

Modal analysis of the models described in Section 5.7 is performed considering a fixed-base condition. This section presents the results in terms of cumulative modal mass participation factor versus frequencies, which shows the global building dynamic response.

Figure 5-6 shows the modal analysis results from the models with RC walls and slabs. The effect of reducing the out-of-plane flexural stiffness by half (uncracked EI_{cr}

model) is a shift in the main building frequencies, particularly in the X and Z (vertical) directions, with respect to the uncracked gross model. In effect, uncracked EI_{cr} model results in a softer structure in comparison to the uncracked gross model. Also, the main mode in the X direction near 7.2 Hz, is split in two, with frequencies about 6.6 Hz and 8 Hz.

The cracked model shows a significant reduction of the main building frequencies. The modal responses in the vertical direction from uncracked EI_{cr} and cracked models are similar for frequencies lower than about 9.5 Hz. This demonstrates the vertical response in this frequency range is driven by out-of-plane flexure of the walls and roof. Above 9.5 Hz, the vertical response of these two models differs, implying the vertical response of the cracked model is mainly due to the in-plane axial stiffness of the walls for this frequency range.

Figure 5-7 shows the modal analysis results from the models with SC walls and RC roof. The modal analysis results are similar to the ones from the models with all RC members. However, the models with SC walls are slightly stiffer, as demonstrated by the increase in the modal frequencies, even though the SC wall thickness is just 4 ft. This analysis shows the contribution of the faceplates to the member stiffness is significant.

Figure 5-6 Effective modal mass participation fraction versus frequency, models with reinforced concrete walls and slabs

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}}2(a),(c)

Figure 5-7 Effective modal mass participation fraction versus frequency, models with steel-plate composite walls and reinforced concrete slabs

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}}2(a),(c)

5.6.3 Test Models using Method 2 for Implementing Effective Stiffness of Steel-Plate Composite walls

Section 5.5.2.2 describes Method 2 for implementing the effective stiffness of SC walls in finite element models. This section includes test models showing the implementation using Method 2 for the SC walls.

The test models consist of a box-type structure modeled in ANSYS using solid-shell (SOLSH190) elements, as shown in Figure 5-8. The walls in this prototype structure have similar aspect ratios to the walls in the representative SMR Reactor Building. The short horizontal dimension (Y) is 60 ft, the long dimension (X) is 120 ft, and the height (Z) is 65 ft.

The model, shown in Figure 5-8, is the same as used in Section 5.6 (see Figure 5-5) except for using Method 2 for the SC walls with no change for the RC roof. The SOLSH190 finite element mesh of for the SC walls is changed from 5-ft thick to the actual SC wall thickness equal to 4-ft thick. The RC roof and mesh are still 5-ft thick. The mesh size was also reduced from 5 ft to 4 ft.

For comparison of shell to solid elements, a test model was constructed using SHELL181 elements having model thickness calculated by Method 1 (see Section 5.5.2.1). The geometry of the structure is the same as the solid element model. Nodes on the 4-ft thick wall and 5-ft thick roof are located at the mid-plane of the solid elements.

Figure 5-8 Box-type building structure modeled by SOLSH190 elements

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}}2(a),(c)

Concrete properties for the roof are the same as used for Method 1. The elasticity modulus is $E_c = 4.102 \times 10^5 kip/ft^2$, the shear modulus is $G_c = 1.754 \times 10^5 kip/ft^2$ and Poisson's ratio $\upsilon_c = 0.17$.

The SC walls have the same concrete properties as used for the roof. The SC walls have 3/4-in.-thick outer plates (faceplates).

Density of the SC walls is taken as 0.5 kip/ft^3 as in the example problem for Method 1, whereas the density ω_c of the RC roof is same as used for Method 1.

Analyses are performed for following two variations:

1. For the SC walls gross, (uncracked) stiffness values are used except for the out-of-plane flexure, in which the effective stiffness from Table 5-1 is used, and considering operational thermal condition. For the RC roof, gross (uncracked) stiffness values are used.
2. For the SC walls gross, effective stiffness values from Table 5-1 for the cracked case, and operational thermal condition are used. For the RC roof, cracked stiffness values are used.

These two variations are the same as considered for SC wall using Method 1. For comparison, these two cases were also analyzed using SHELL181 elements with model thickness set by Method 1.

For the test problems described in this section, the model thickness of both SC walls and RC roof are the actual wall and roof thickness, respectively. Material properties are as follows:

Material properties for cracked reinforced concrete roof

The model thickness of the RC roof is the same as the actual roof thickness. Material properties are obtained from Equation 5-4 through Equation 5-7, using EI_{eff} = 0.5EI_{gross}, EA_{eff} = 0.5EA_{gross} and GA_{eff} = 0.5GA_{gross}. The resulting values are:

$$\begin{aligned} t_m &= 5ft \\ E_m &= 0.5E_c \\ G_m &= 0.5G_c \\ \gamma_m &= w_c \end{aligned}$$

Material properties for uncracked steel-plate composite walls using Method 2 and SOLSH190 elements

Material properties are calculated considering 3/4-in. faceplates and the operating thermal condition. The uncracked case is an example of a case where the outer layer thickness is taken as greater than the actual faceplate thickness. The model thickness of the SC walls is the same as the actual wall thickness, i.e., 4 ft.

The thickness of the outer layers is assigned to be twice the thickness of the faceplates

$$t_p' = 2t_p = 2 \times \frac{0.75}{12} = 0.125\text{ft}$$

and the thickness of the middle layer is

$$t_p' = t_{sc} - 2t_p' = 3.75\text{ft}$$

Material properties for the operating condition with uncracked in-plane shear are:

$$\begin{aligned} E_{mc} &= 1.3598E_c \\ E_{ms} &= 0.0188E_s \\ G_{mc} &= 0.8332G_c \end{aligned}$$

$$G_{ms} = 0.8332 G_s$$

$$\Upsilon_m = 0.500 kip/ft^3$$

Material properties for cracked steel-plate composite walls using Method 2 and SOLSH190 elements

Material properties are calculated considering 3/4-in. faceplates and the operating thermal condition. The cracked case is an example where the outer layer thickness is the same as the actual faceplate thickness. The model thickness of the SC walls is the same as the actual wall thickness, i.e., 4 ft.

The thickness of the outer layers is assigned to be the thickness of the faceplates

$$t_p = \frac{0.75}{12} = 0.0625\,\text{ft}$$

and the thickness of the middle layer is

$$t_c = t_{sc} - 2t_p = 3.875\,\text{ft}$$

Material properties for the operating condition with cracked in-plane shear are:

$$E_{mc} = 0.6179 E_c$$

$$E_{ms} = 0.6407 E_s$$

$$G_{mc} = 0.4576 G_c$$

$$G_{ms} = 0.4576 G_s$$

$$\Upsilon_m = 0.500 kip/ft^3$$

Material properties and thickness for uncracked steel-plate composite walls using Method 1 and SHELL181 elements

A single layer model with thickness and material properties from Section 5.5.2.1 is used.

The model thickness and material properties are obtained from Equation 5-4 through Equation 5-7, using EI_{eff} , EA_{gross} and $GA_{eff} = GA_{uncr}$, considering 3/4-in. faceplates and operating thermal condition. The resulting values are:

$$t_m = 3.788 ft$$

$$E_m = 5.576 \times 10^5 kip/ft^2$$

$$G_m = 2.321 \times 10^5 kip/ft^2$$

$$\Upsilon_m = 0.528 kip/ft^3$$

Material properties and thickness for uncracked steel-plate composite walls using Method 1 and SHELL181 elements

A single layer model with thickness and material properties from Section 5.5.2.1 is used.

The model thickness and material properties are obtained from Equation 5-4 through Equation 5-7, using EI_{eff}, EA_{eff} and $GA_{eff} = GA_{cr}$, considering 3/4-in. faceplates and operating thermal condition. The resulting values are:

$$t_m = 4.797 ft$$

$$E_m = 2.746 \times 10^5 kip/ft^2$$

$$G_m = 8.385 \times 10^4 kip/ft^2$$

$$\Upsilon_m = 0.417 kip/ft^3$$

5.6.4 Modal Analysis of Test Models using Method 2 for Implementing Effective Stiffness of Steel-Plate Composite Walls

Modal analysis is performed using the models described in Section 5.6.3, considering a fixed-base condition. This section presents the results in terms cumulative modal mass participation factor versus frequencies, which shows the global building dynamic response.

Figure 5-9, Figure 5-10 and, Figure 5-11 show the cumulative modal mass participation factor versus frequencies, for the east-west (X), north-south (Y), and vertical (Z) directions, respectively. Each figure shows results for uncracked and cracked conditions obtained using the three modeling options: (1) SHELL181 with 1 layer, (2) SOLSH190 with 3 layers and Method 1, and (3) SOLSH190 with three layers and Method 2.

The cumulative mass participation ratios versus frequency calculated using SOLSH190 with both Method 1 and Method 2 are close to each other. Results using the SHELL181 elements are generally in agreement with the SOLSH190 elements; however, frequencies calculated using the SHELL181 elements are slightly lower than with SOLSH190.

In the models using shell elements, the total span of roof and walls is represented by elements spanning between midplane nodes on adjacent walls and slabs (i.e., spans represented in the model are greater than the open span between connection regions). The SOLSH190 element provides a better representation of the connection region. For this test problem, the difference using SHELL181 elements is not large compared to that due to the variation being considered due to cracking. For cases using greater ratios of total to open span, the shell elements include rigid offsets at connections. For models with a mixture of SOLSH190 and SHELL181, the SOLSH190 element can be used to model the connection region and SHELL181 element coupled to the face of the solid element.

Figure 5-9 Cumulative mass participation ratios versus frequency, east-west (X) direction

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}}2(a),(c)

Figure 5-10 Cumulative mass participation ratios versus frequency, north-south (Y) direction

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}}2(a),(c)

Figure 5-11 Cumulative mass participation ratios versus frequency, vertical (Z) direction

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}}2(a),(c)

5.7 Modeling of Steel-Plate Composite and Reinforced Concrete Walls Summary

A methodology is presented to implement code-specified, effective stiffness values to ANSYS finite element models. Both uncracked and cracked conditions are included, as well as RC members and SC walls.

Tests models show the application of the methodology to finite element models used for the seismic analysis of a representative SMR. Two model sets are developed: one set consists of models with all RC members, and the other set consists of models with SC walls and RC slabs. Effective stiffness values specified by ASCE 4-16 and AISC N690-18 are applied to the models considering uncracked and cracked cases.

The modeling of SC and RC walls and slabs can be performed using ANSYS SOLSH190 or SHELL181 elements. Each of these elements is suitable for analyzing thin to moderately-thick shell structures. SOLSH190 elements can be connected directly with other elements and provide a convenient means for modeling of the connection region and open span of walls and slabs. The test cases demonstrate methods that can be used to input section properties for SOLSH190 or SHELL181 elements to produce the effective stiffness values specified by ASCE 4-16 and AISC N690-18.

The use of SOLSH190 elements is recommended because of the following advantages:

- SOLSH190 usually gives more accurate predictions than classical shell theory when the shell is thick.
- The SOLSH190 mesh can use eight-node solid elements to represent the actual wall thickness. The primary advantage of the solid shell elements is they facilitate connection with other continuum elements, such as contact and fluid elements. The SOLSH190 elements provide a three-dimensional representation of the member connection region and simplify the generation of the finite element mesh from CAD or solid models of the structure.
- For use in design, shell member forces and moments per unit length are automatically calculated for SOLSH190 elements. There can be cases where meshes made using only using SOLSH190 elements have some open spans with multiple layers of elements through the thickness. This complicates the post-processing scripts used to obtain forces and moment for design. For these cases, the open span can be represented accurately using a single layer of SHELL181 elements coupled to the face of the connection represented by solid elements.

Two alternate methods can be used for implementing the effective stiffness values using three-layered SOLSH190 elements. In both methods, the equivalent material properties are orthotropic. The methods differ in layer thickness and material properties; however, both result in section stiffnesses equal to the effective section stiffnesses defined by AISC N690-18, Section N9.2.2.

Method 1 is described in Section 5.5.2.1. In Method 1, a single material is used for the middle layer. The outer two layers are dummy layers with zero density and insignificant value of Young's modulus and shear modulus, e.g. Young's Modulus, $E \leq 10$ kip/ft^2, Poisson's Ratio V=0.17, and shear modulus G=E/2(1+V).

Method 2 is described in Section 5.5.2.2 and is applicable to SC walls only. In Method 2, the thicknesses are equal to or greater than the actual SC faceplate thickness.

Recommendations for choosing Method 1 versus Method 2 are:

- For RC members, only Method 1 is applicable.
- For SC walls with uncracked stiffness, Method 2 is recommended. The three layers used in Method 2 are a more realistic representation of the actual steel and uncracked concrete layer thicknesses.
- For SC walls, Method 1 can result in a middle layer model thickness greater than the actual SC wall thickness. This effect becomes more pronounced as the faceplate thickness increases. The mesh must then be reconstructed to accommodate the thick middle layer. Therefore, Method 1 is not recommended for SC walls for any case where the model thickness is greater than the actual wall thickness, because it requires remeshing that eliminates the advantages of using solid shell elements.
- For SC walls, Method 2 is recommended except in cases where an assumption that the outer layers are the same thickness as the SC wall faceplates results in a negative value for Young's modulus of outer layers. This can be observed in walls where the faceplate thickness is relatively thin, (i.e., a 4-ft.-thick, uncracked wall with faceplate thickness equal to 3/4-in. or less). For these cases, Method 1 is preferable. In these cases where Method 1 is used, the effective thickness of the middle layer is less than the wall thickness and dummy layers are used so remeshing of the model is not needed.

6.0 Design Methodology for Steel-Plate Composite Walls

This section develops the strength design methodology for application of SC walls. The design methodology is based on the requirements of ANSI/AISC N690-18 (Reference 10.1.3) and ANSI/AISC 360-16 (Reference 10.1.15), as required by ANSI/AISC N690-18.

Design and detailing of the SC wall connections are included in Section 7.0 of this document.

Mitigation of corrosion effects will be tailored to site-specific factors. In order to address design for corrosion effects, NuScale plans to implement a defense-in-depth approach depending on the severity of site-specific environmental conditions as well as exposure and performance requirements of SC walls.

For SC walls in a below-grade external environment, also considering the possibility of plant life extension of up to 80 years, a graded approach is followed as inservice inspection and inservice repair of these components would be impractical. Exterior SC walls below grade are designed to be protected from corrosion using a graded approach as follows:

- As a minimum, a coal tar epoxy system coating specifically suited for below-grade protection of carbon steel.
- Additional protection such as Controlled Low Strength Material or shotcrete may be employed as a cementitious material only for environments with high chloride or hydrogen sulfide.
- Finally, backfill with controlled pH and chloride limits governed by site-specific conditions is placed and thoroughly compacted to reduce impact from corrosive properties of soil.

For SC walls in an above-grade external environment, a coating specifically suited for above-grade protection of carbon steel is applied. If additional protection is deemed necessary, a concrete coating, or vinyl or aluminum siding may be employed.

6.1 Applicability

The methodology applies to straight SC walls (refer to note in Section N9.1 of AISC N690-18 and its Commentary), SC walls with steel-headed stud anchors (refer to Section 6.3.3.1), and structures designed using load and resistance factor design.

The methodology includes requirements and rules for designing and detailing SC walls, including response and sizing requirements under impactive and impulsive loading (refer to Section 6.7).

6.2 Background

In design, SC walls provide better resistance to blast and earthquake events, higher ultimate strength, and higher ductility. In addition, the modular construction of SC walls optimizes schedule and labor requirements.

In SC construction, the concrete walls are reinforced with two steel faceplates attached to concrete using steel anchors, such as steel-headed stud anchors to ensure composite behavior, and connected to each other using steel tie bars for integrity as illustrated in Figure 6-1.

Figure 6-1 Typical steel-composite wall configuration

{{

}}2(a),(c)

6.3 Requirements for designing steel-composite walls

The requirements for designing SC walls are summarized in the subsections below. The requirements are categorized into dimensional and material requirements, as well as slenderness, stability, and strength requirements.

According to Section N9.2 of AISC N690-18, for design purposes, SC walls are divided into interior regions and connection regions.

Connection regions consist of perimeter strips with a width not less than the SC wall thickness, t_{sc}, and not more than twice the SC wall thickness, $2t_{sc}$. Figure 6-2 illustrates the typical interior and connection regions for SC walls.

Figure 6-2 Interior versus connection regions for steel-plate composite wall design

{{

}}$^{2(a),(c)}$

6.3.1 Dimensional and Material Property Requirements

The dimensional and material property requirements for the design of SC walls are specified in Table 6-1. The table also includes values recommended for the initial design cycle. These values are to be later calibrated or modified during the design process to satisfy the requirements for the design case in hand.

Per Section N9.1.1(e) of AISC N690-18, lightweight concrete is not used for the construction of SC walls.

Table 6-1 Dimensional and material properties requirements and recommendations for steel composite walls

Parameter	Minimum Value	Maximum Value	Notes or Values Recommended for Initial Design Values
t_{sc} (in.) for exterior wall	{{		{{ }}2(a),(c)
t_{sc} (in.) for interior wall			{{ }}2(a),(c)
t_p (in.)			{{ }}2(a),(c) (thickness for both faceplates is to be same)
$\rho = 2t_p/t_{sc}$			{{ }}2(a),(c) (based on t_p and t_{sc} values above)
F_y (ksi)			{{ }}2(a),(c) (F_y for both faceplates is to be the same)
f'_c (ksi)		}}2(a),(c)	Specified concrete compressive strength is {{ }}2(a),(c) ksi for exterior walls above grade and {{ }}2(a),(c) ksi elsewhere
w_c (lb/ft^3)	---	---	Weight of concrete per unit volume of {{ }}2(a),(c) is recommended
ν_c	---	---	{{ }}2(a),(c) is recommended
Anchor spacing (in.)	Maximum and minimum values are governed by design rules in the subsequent sections		$t_{sc}/6$
Tie spacing (in.)			Tie spacing ≤ $t_{sc}/2$ (maximum spacing of the ties is to be limited to half the wall thickness to maximize out-of-plane shear strength)

6.3.2 Faceplate Slenderness Requirements

Local buckling of the faceplate between the steel anchors is an important limit state to be considered when designing SC walls. This requirement is to ensure yielding in compression controls rather than local buckling of the faceplate between ties and anchors.

The faceplates of the SC walls are designed to be nonslender. This property is achieved by providing sufficient anchorage to the concrete through anchors and ties, thereby satisfying Equation A-N9-2 of Appendix N9 of AISC N690-18, as shown in Equation 6-1 below

$$\frac{b}{t_p} \leq 1.0\sqrt{\frac{E_s}{F_y}} \qquad \text{Equation 6-1}$$

where the largest unsupported length b is the minimum of spacing between shear reinforcement (i.e., steel-headed anchors and ties) in the horizontal and vertical directions.

6.3.3 Requirements for Steel Anchors and Ties

Requirements for the steel anchors and ties presented in this section ensure composite action between the concrete and the faceplates as required by Section N.9.1.1 (g) of AISC N690-18.

6.3.3.1 Steel Anchors

The use of steel-headed stud anchors is required in the design of SC walls to avoid the need for testing. These anchors are classified as yielding steel anchors.

Classification and available strength for other types of anchors are to be established by testing according to Section N.9.1.4a of Appendix N9 of AISC N690-18 and are outside the scope of this document.

Steel anchor diameter requirement

The maximum allowable diameter of the headed stud anchors used to transfer shear only between the faceplates and concrete is 1 inch. Also, the diameter of the headed anchor studs is to satisfy Section I8.1 of AISC 360-16, (Reference 10.1.15) as shown below:

$$d_s \leq \min(2.5t_p, 1.0\text{in}) \qquad \text{Equation 6-2}$$

Steel anchor head diameter requirement

Steel-headed stud anchors subject to tension or interaction of shear and tension have a diameter of the head greater than or equal to 1.6 times the diameter of the shank according to Section I8.3 of AISC 360-16. That is:

$$d_{sh} \geq 1.6d_s \qquad \text{Equation 6-3}$$

Steel anchor length requirement

Steel-headed anchors in SC walls are subject to an interaction of shear and tension. Thus, the length of these anchors is to be not fewer than eight stud diameters from the base of the stud to the top of the stud head after installation. This requirement is per Section I8.3 of AISC 360-16, (Reference 10.1.15). That is:

$$h_{an} \geq 5d_s \qquad \text{Equation 6-4}$$

Available shear strength of steel anchors

The available shear strength for the headed stud anchors Q_{cv} is per Equation I8.3 of AISC 360-16, as given in Equation 6-5 below.

$$Q_{cv} = 0.65F_{ua}A_{sa} \quad \text{Equation 6-5}$$

Steel anchor spacing requirements

Spacing between the steel anchors is to not exceed the minimum of (s_1, s_2). The resulting spacing, s_3, is within the limits of Section I8.3e of AISC 360-16 as given below.

s_1 - Spacing required to develop the yield strength of the faceplate over the development length L_d given as:

$$s_1 \leq c1\sqrt{\frac{Q_{cv}L_d}{T_p}} \quad \text{Equation 6-6}$$

where $L_d \leq 3t_{sc}$ and $T_p = F_y t_p$, $c1$=1.0 for yielding steel anchor.

Steel anchor spacing is typically governed by the requirement of development length being less or equal $3t_{sc}$; however, for portions of the SC wall with large out-of-plane moment, the steel anchor spacing can be controlled by Equation 6-7 to achieve interfacial shear strength greater than the available out-of-plane shear strength, V_c, determined as explained below.

s_2 - Spacing required to prevent interfacial shear failure before out-of-plane shear failure of the SC section given as:

$$s_2 \leq c1\sqrt{\frac{Q_{cv}l}{V_c/0.9t_{sc}}} \quad \text{Equation 6-7}$$

where l = 12 in./ft (unit width) and $c1$= 1.0 for yielding steel anchor

As mentioned above, this requirement does not typically govern the design. This requirement is to be verified after the available out-of-plane shear strength is calculated.

The resulting spacing is to be within the limits of Section I8.3e of AISC 360-16, in any direction.

$$4d_s \leq s_3 \leq 32d_s \quad \text{Equation 6-8}$$

Steel anchor lateral clear concrete cover requirements

Steel anchors have at least 1.5 in. lateral clearance when concrete is not exposed to weather and 2 in. of clearance when the concrete is exposed to earth or weather. This is based on Section 7.7.1 of ACI-318.

6.3.3.2 Ties

The opposite faceplates of the SC walls are connected to each other using ties consisting of individual components, such as structural shapes, frames, or bars. A tie can be a single element (i.e. tie rod) or several structural elements (e.g., tie bar with gusset plate at one or both ends).

The ties provide structural integrity to prevent section splitting and serve as out-of-plane shear reinforcement.

Ties Spacing Requirements

Spacing between ties in the horizontal and vertical directions is governed by the requirement of N9.1.5 of AISC N690-18 as shown below:

$$S_t \leq t_{sc} \qquad \text{Equation 6-9}$$

However, for the specific wall thicknesses anticipated, the recommended tie spacing is less than or equal to half of the wall thickness to maximize the out-of-plane shear strength.

Classification of Ties

Per Section N9.1.5a of AISC N690-18, the ties are classified as yielding shear reinforcement if:

$$F_{ny} \leq 0.8F_{nr} \qquad \text{Equation 6-10}$$

where F_{nr} is the minimum of the nominal rupture strength of the tie or the nominal strength of the associated connection of the tie to the faceplate (welded or bolted), whichever is smaller (kips).

For the cases when complete joint penetration groove welds are used for the connection between the ties and the faceplate, F_{nr} is the minimum nominal rupture strength of the tie, because the nominal strength of the connection is equal or greater than the member strength.

Required and Available Tensile Strength for Ties

The required tensile strength of individual ties is calculated as

$$F_{req} = \left(\frac{t_p F_y t_{sc}}{4}\right)\left(\frac{S_{tt}}{S_{tl}}\right)\left[\frac{6}{18\left(\frac{t_{sc}}{S_{tl}}\right)^2 + 1}\right] \quad \text{Equation 6-11}$$

The available strength for the ties and their connections to the faceplate are calculated and compared with the required tensile strength from Equation 6-11. The design tensile strength of the tie is calculated as the minimum (ϕF_{ny}, ϕF_{nr}), where ϕF_{ny} and ϕF_{nr} are calculated based on Section J4.1 of AISC 360-16 and given in Equation 6-12 and Equation 6-13:

- For tensile yielding of the tie:

$$\phi F_{ny} = 0.9 F_{yt} A_{tie} \quad \text{Equation 6-12}$$

- For the tensile rupture of the tie, the effective net area of the tie can be taken equal to the net area:

$$\phi F_{n\mathrm{r}} = 0.75 F_{ut} A_{ntie} \quad \text{Equation 6-13}$$

6.4 Finite Element Analysis Model Requirements

This section discusses the general and stiffness requirements for analyzing SC walls using finite element analysis (FEA). Modeling requirements for openings are based on Section N9.1.7 of AISC N690-18 and its Commentary. These requirements are discussed in Section 6.8.

The general FEA requirements are:

- Steel-plate composite walls are analyzed using elastic, three-dimensional thick shell or solid elements.
- When using shell elements to model the expanse of an SC wall, use at least four elements along the short side and six to eight elements along the long side to capture local modes of vibration in accordance with Steel Design Guide 32, (Reference 10.1.5).
- Viscous damping ratio does not exceed {{ }}[2(a),(c)] for SSE for the determination of required strength for SC walls.
- For analysis involving accident thermal loads, heat transfer analysis is conducted to estimate the temperature time histories and through-section temperature profiles produced by the thermal accident conditions. Refer to Section 6.4.3 for material properties modeling requirements.
- Typically SC walls related to nuclear facilities satisfy the stability requirements. Stability is checked to ensure that second-order analyses are not needed. According

to Section N9.1.2b, second-order analysis need not be performed if Equation 10-7 of Section 10.10.1(b) of ACI 318-08 is satisfied as shown in Equation 6-14 below.

$$\frac{Kl_u}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right) \leq 40 \qquad \text{Equation 6-14}$$

6.4.1 Loads and Load Combinations

Loads acting on the SC walls are determined based on Chapter NB2 of AISC N690-18, as shown in Table 6-2 below.

Table 6-2 Load combinations

Combination Level	Combination
Normal	1.4(D + R_o + F) + To + C
	1.2(D + R_o + F) + 1.6(L + H) + 0.5(L_r or S or R) + 1.2 T_o + 1.4C
	1.2(D + R_o + F) + 1.6(L_r or S or R) + 0.8(L + H) + 1.2 T_o + 1.4C
Severe environmental	1.2 (D + F + R_o) + W + 0.8L + 1.6H + 0.5(L_r or S or R) + T_o + C
Extreme environmental and abnormal	D + 0.8L + C + T_o + Ro + Ess + F + H
	D + 0.8L + T_o + R_o + Wt + F + H
	D + 0.8L + C + 1.2P_a + R_a + T_a + F + H
	D + 0.8L + (P_a + R_a + T_a) + (Y_r + Y_j + Y_m) + 0.7E_{ss} + F + H

6.4.2 Stiffness Modeling Requirements

6.4.2.1 Effective Flexural Stiffness

For the analysis of the SC walls, the effective flexural stiffness of the wall is based on the stiffness of the cracked transformed section, including the faceplates and the cracked concrete infill. The effective flexural stiffness is calculated according to Section N9.2.2 (a) of AISC N690-18 as depicted in Equation 6-15.

For the thermal accident condition, Equation 6-15 takes into account the reduction in flexural stiffness due to additional concrete cracking resulting from the thermal accident condition.

For the operating thermal condition, ΔT_{savg} = 0 is used in Equation 6-15.

ΔT_{savg} is the average of the maximum surface temperature increases for the faceplate due to accident thermal conditions.

$$EI_{eff} = (E_s I_s + c_2 E_c I_c)\left(1 - \frac{\Delta T_{savg}}{150}\right) \geq E_s I_s \qquad \text{Equation 6-15}$$

where,

$$E_c = w_c^{1.5}\sqrt{f'_c} \quad ,$$

$$I_c = l\frac{t_c^3}{12},$$

$$I_s = lt_p\frac{(t_{sd}-t_p)^2}{2},$$

$l = 12in/ft$, and

$$c_2 = 0.48\frac{2E_s t_p}{E_c t_{sc}} + 0.10 .$$

6.4.2.2 Effective In-Plane Shear Stiffness

For the thermal accident condition, the effective in-plane shear stiffness per unit width is taken for the cracked section as calculated in Equation 6-17.

For operating conditions, the effective in-plane shear stiffness depends on the ratio of the average in-plane shear demand, S_{rxy}, to the concrete cracking threshold, S_{cr}. The effective in-plane shear stiffness is calculated per Section N9.2.2 (b) of AISC N690-18. The equations for calculating the effective in-plane shear stiffness are given below.

- If $S_{rxy} \le S_{cr}$

$$GA_{eff} = GA_{uncr} = GA_s + G_cA_c \qquad \text{Equation 6-16}$$

- If $S_{rxy} > S_{cr}$

$$GA_{eff} = GA_{cr} = 0.5\bar{\rho}^{-0.45}GA_s \qquad \text{Equation 6-17}$$

- If $S_{cr} \le S_{rxy} < 2S_{cr}$

$$GA_{eff} = GA_{uncr} - \left(\frac{GA_{uncr} - GA_{cr}}{S_{cr}}\right)(S_{rxy} - S_{cr}) \qquad \text{Equation 6-18}$$

where the strength adjusted reinforcement ratio $\bar{\rho}$ is calculated as:

$$\bar{\rho} = \frac{A_s F_y}{31.6 A_c \sqrt{f'_c}} \qquad \text{Equation 6-19}$$

The concrete cracking threshold per unit width is calculated as

$$S_{cr} = \frac{0.063\sqrt{f'_c}}{G_c} G A_{uncr} \qquad \text{Equation 6-20}$$

6.4.3 Geometric and Material Properties for Finite Element Analysis

The following geometric and material properties are to be used in the finite element model per Section N9.2.3 of AISC N690-18. Poisson's ratio ν_m, thermal expansion α_m, and thermal conductivity κ_m of the concrete are used to model the SC wall.

6.4.4 Additional Required Out-of-Plane Flexural Strength for the Accident Thermal Condition

Heat transfer analysis is performed for accident thermal conditions to estimate temperature histories and the through-section temperature profile.

6.5 Required Strength Determination

Strength demand for in-plane membrane forces, out-of-plane moments, and out-of-plane shear forces is determined by averaging demand over panel sections. The width and length of the panel sections are determined based on Section N9.2.5 of AISC N690-18 as follows.

- Away from openings and connection regions, the panel width and height does not exceed twice the section thickness.
- In the vicinity of openings and in connection regions, the panel width and height do not exceed the section thickness.

For each demand type, the required strength for each panel sections of SC walls is denoted as follows, where x and y are the local coordinate axes in the plane of the wall associated with the finite element model:

M_{rx} and M_{ry} are the required out-of-plane flexural strength per unit width in direction x and y, respectively (kip-in./ft)

M_{rxy} is the required twisting moment strength per unit width (kip-in./ft)

S_{rx} and S_{ry} are required membrane axial strength per unit width in direction x and y (kip/ft)

S_{rxy} is the required membrane in-plane shear strength per unit width (kip/ft)

V_{rx} and V_{ry} are the required out-of-plane shear strength per unit width along edge parallel to the directions x and y, respectively.

6.6 Available Strength Calculation

6.6.1 Tensile Strength

The available tensile strength of the SC wall $\phi_t P_{n,t}$ is governed by the resistance of the faceplates. The tensile strength contribution of concrete infill is ignored, along with the contribution of steel ribs to the available strength of SC walls. The available uniaxial tensile strength per unit width of the panel section is determined based on Chapter D of AISC 360-16, as follows:

- For sections with no openings in faceplates:

$$\phi_t P_{n,t} = \min(\phi_t P_{n,y}, \phi_t P_{n,u}) \quad \text{Equation 6-21}$$

where $\phi_t P_{n,y}$ is the tensile yielding of the gross section given as

$$\phi_t P_{n,y} = 0.9 F_y A_s \quad \text{Equation 6-22}$$

and $\phi_t P_{n,u}$ is the tensile rupture of the net section given in Equation 6-23. The effective area of the faceplate is taken to be equal to the net area:

$$\phi_t P_{n,u} = 0.75 F_u A_{sn} \quad \text{Equation 6-23}$$

- Per Section N9.3.1 of AISC N690-18, for sections with openings in the faceplates, Equation 5-25 is to be satisfied:

$$\phi_t P_{n,u} > \phi_t P_{n,y} \quad \text{Equation 6-24}$$

6.6.2 Compressive Strength

The available compressive strength per unit width of the panel section $\phi_c P_{n,comp}$ is calculated per Section I2.1b of AISC 360-16 as modified in Section N9.3.2 of AISC N690-18.

The nominal compressive strength per unit length P_{no} is calculated as

$$P_{no} = F_y A_{sn} + 0.85 f'_c A_c \quad \text{Equation 6-25}$$

The elastic critical buckling load per unit length is calculated as given in Equation 6-26, where L is the laterally unbraced length of the panel section:

$$P_e = \frac{\pi^2 EI_{eff\_buck}}{L^2} \quad \text{Equation 6-26}$$

and the effective panel section stiffness per unit width for buckling evaluation is calculated as:

$$EI_{eff\_buck} = E_s I_s + 0.6 E_c I_c \quad \text{Equation 6-27}$$

The available compressive strength per unit width $\phi_c P_{n,comp}$ is calculated as:

- If $P_{no}/P_e \leq 2.25$

$$\phi_c P_{n,\,comp} = 0.75 P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right] \quad \text{Equation 6-28}$$

- If $P_{no}/P_e > 2.25$

$$\phi_c P_{n,\,comp} = 0.75(0.877 P_e) = 0.65775 P_e \quad \text{Equation 6-29}$$

6.6.3 Out-of-Plane Flexural Strength

The available flexural strength per unit width $\phi_b M_n$ is determined using equation C-A-N9-11 of Section N9.3.3 of the Commentary as shown in Equation 6-30. This equation is used in lieu of the more conservative equation given in Section N9.3.3 of AISC N690-18.

$$\phi_b M_n = 0.9\left[F_y(A_s^F)(t_{sc} - t_p) - \frac{1}{2} f l c_c\left(\frac{c_c}{3} + \frac{t_p}{2}\right)\right] \quad \text{Equation 6-30}$$

$$C_c = 2t_p\left(\frac{F_y}{f_c'} - n\right) \geq 0 \quad \text{Equation 6-31}$$

$$f = \min\left(\frac{F_y}{n}, f_c'\right) \quad \text{Equation 6-32}$$

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

6.6.4 In-Plane Shear Strength

The available in-plane shear strength unit width of the panel section $\phi_{vi}V_{ni}$ is calculated per Section N9.3.4 of AISC N690-18 and shown in Equation 6-33.

$$\phi_{vi}V_{ni} = 0.90\kappa F_y A_s \quad \text{Equation 6-33}$$

$$\kappa = 1.11 - \frac{5.16 A_s F_y}{31.6 A_c \sqrt{f_c'}} \leq 1.0 \quad \text{Equation 6-34}$$

6.6.5 Out-of-Plane Shear Strength

Per Section N9.3.5 of AISC N690-18, the nominal out-of-plane shear strength $\phi_{no}V_{no}$ per unit width (l =12 in./ft) is established using the provisions of Section N9.3.5 of AISC N690-18. The rules in this section are limited to the case when the spacing of the ties is less than or equal to half of the SC wall thickness to maximize out-of-plane strength. Otherwise, refer to Section N9.3.5. The nominal out-of-plane shear strength $\phi_{no}V_{no}$ is calculated as:

$$\Phi_{vo}V_{no} = 0.75(V_{conc} + V_s) \quad \text{Equation 6-35}$$

where V_{conc} is per Equation 6-36 and V_s is per Equation 6-37:

$$V_{conc} = 0.05\sqrt{f_c'}\, t_c l \quad \text{Equation 6-36}$$

$$V_s = \xi F_{t} \frac{t_c l}{s_{tl} s_{tt}} \leq 0.25\sqrt{f_c'}\, t_c l \quad \text{Equation 6-37}$$

$\xi = 1.0$ for yielding shear reinforcement and 0.5 for non-yielding shear reinforcement. Classification of ties is determined using Equation 6-10.

6.6.6 Combined Forces - Out-of-Plane Shear Forces

The interaction of out-of-plane shear forces is evaluated using Section N9.3.6a of AISC N690-18. This section is specific to the case when out-of-plane shear reinforcement is spaced no greater than half the section thickness. Refer to Section N9.3.6 of AISC- N690-18 for other cases. The interaction of out-of-plane shear forces is only required when both the required out-of-plane shear strength per unit width along local x and y directions, V_{rx} and V_{ry}, respectively, are greater than the available out-of-plane shear strength contributed by the concrete per unit length, that is, if (V_{rx} and V_{ry}) > $0.75V_{conc}$.

$$\left[\left(\frac{V_r - 0.75V_{conc}}{\varnothing_{vo}V_{no} - 0.75V_{conc}}\right)_x + \left(\frac{V_r - 0.75V_{conc}}{\varnothing_{vo}V_{no} - 0.75V_{conc}}\right)_y\right]^{5/3} + \left[\frac{\frac{\sqrt{v_{rx}^2 + v_{ry}^2}}{0.9t_{sc}}}{\Psi\left(\frac{lQ_{cv}^{avg}}{s^2}\right)}\right]^{5/3} \leq 1.0 \qquad \text{Equation 6-38}$$

where V_r is the required out-of-plane shear strength per unit width of the SC panel section in local x (V_{rx}) and y (V_{ry}) directions using load and resistance factor design load combinations (kip/ft)

$\phi_{vo}V_{no}$ (kip/ft) is the available out-of-plane shear strength per unit width of the SC panel section in the local x and y directions as calculated in Equation 6-35

V_{conc} is per Equation 6-36 (kip/ft)

s is the spacing of the steel anchors (in.)

Ψ = 1.0 for sections with yielding ties and anchors, Ψ =0.5 for sections with either non-yielding ties or steel anchors.

Q_{cv}^{avg} is the weighted average of the available interfacial shear strength of ties and steel anchors (kip). This value is calculated based on Section N9.3.6a of the Commentary of AISC N690-18 as follows:

$$Q_{cv}^{avg} = \frac{\mathrm{n_{et}}Q_{cv}^{tie} + \mathrm{n_{es}}Q_{cv}}{\mathrm{n_{et}} + \mathrm{n_{es}}} \qquad \text{Equation 6-39}$$

where Q_{cv}^{tie} and Q_{cv} are calculated per Equation 6-5 for both the ties and the anchors.

n_{et} and n_{es} are the effective number of ties and steel anchors contributing to a unit cell calculated as explained in Section N9.3.6a of the Commentary of AISC N690-18 and illustrated below.

The unit cell is the quadrilateral region between four ties shown in Figure 6-3. As shown in the figure, only a quarter of the ties at each corner are contributing to the strength of the unit cell. That makes n_{et} = 1. The steel anchors inside the unit cell contribute with full strength; the steel anchors on the unit cell boundary contribute 50 percent of their strength to the unit cell. This makes n_{es} = 15.

Figure 6-3 Unit cell for calculating weighted average of the available interfacial shear strength of tie and shear anchor

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}}2(a),(c)

6.6.7 Combined Forces - 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}) are checked using interaction equations based on a conservative simplified design approach (refer to Figure 6-4) consisting of:

- Dividing the SC panel section into two notional halves, each consisting of one faceplate and half of the concrete infill thickness
- Calculating the required in-plane strengths (S_{rx}', S_{ry}', and S_{rxy}') for each notional half
- Calculating the required in-plane principal strengths ($S_{r,max}$ and $S_{r,min}$) for each notional half as shown in Section N9.3.6b of the Commentary of AISC N690-18 and shown in Figure 6-4

Figure 6-4 Combined forces acting on a panel section and notional halves

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}}2(a),(c)

The interaction equation is calculated as given in Section 9.3.6b of AISC N690-18 and described below.

- For each SC panel section, calculate parameters used for distributing required flexural strength into the corresponding membrane couples acting on each notional half of the SC panel section, j_x, j_y, j_{xy} as shown below.
  - For j_x
    - j_x = 0.9 if $S_{rx} > -0.6P_{no}$ (tensions dominated)
    - j_x = 0.67 if $S_{rx} \leq -0.6P_{no}$ (compression dominated)

    where P_{no} is per Equation 6-25,
  - For j_y
    - j_y = 0.9 if $S_{ry} > -0.6P_{no}$
    - j_y = 0.67 if $S_{ry} \leq -0.6P_{no}$

    where P_{no} is per Equation 6-25
  - j_{xy}=0.67
- The required membrane axial strength per unit width in directions x and y for each notional half of the SC panel section are calculated as shown below and in Figure 6-4 (kip/ft)

$$S_{rx}' = \frac{S_{rx}}{2} \pm \frac{M_{rx}}{j_x t_{sc}}$$ Equation 6-40

$$S_{ry}' = \frac{S_{rx}}{2} \pm \frac{M_{ry}}{j_y t_{sc}}$$ Equation 6-41

- The required membrane in-plane shear strength per unit width for each notional half of the SC panel section as shown below and in Figure 6-4 (kip/ft)

$$S_{ry}' = \frac{S_{rxy}}{2} \pm \frac{M_{rxy}}{j_{xy} t_{sc}}$$ Equation 6-42

- Two methods are called in Section 9.3.6b of AISC N690-18 to calculate the interaction ratio. One of these methods is listed here. Refer to Section 9.3.6b for the other method.

The interaction ratio is calculated as follows:

- Calculate the maximum and minimum required principal in-plane strengths per unit width for each notional half of the SC panel section $S_{r,max}$ and $S_{r,min}$ as shown below.

$$S_{r,\,max}, S_{r,\,min} = \frac{S_{rx}' + S_{ry}'}{2} \pm \sqrt{\left(\frac{S_{rx}' - S_{ry}'}{2}\right)^2 + (S_{rxy}')^2}$$ Equation 6-43

- For each notional half, the interaction is limited by one of the three interaction equations shown below as appropriate:
  - For $S_{rmax} + S_{rmin} \geq 0$

$$\alpha\left(\frac{S_{r,\,max} + S_{r,\,min}}{2V_{ci}}\right) + \left(\frac{S_{r,\,max} - S_{r,\,min}}{2V_{ci}}\right) \leq 1.0$$ Equation 6-44

  - For $S_{rmax} > 0$ and $S_{rmax} + S_{rmin} < 0$

$$\frac{S_{r,\,max}}{V_{ci}} - \beta\left(\frac{S_{r,\,max} + S_{r,\,min}}{V_{ci}}\right) \leq 1.0$$ Equation 6-45

  - For $S_{rmax} \leq 0$ and $S_{rmin} \leq 0$

$$-\beta\left(\frac{S_{rmin}}{V_{ci}}\right) \leq 1.0$$ Equation 6-46

Parameters used in Equation 6-44 through Equation 6-46 are as follows:

$$\alpha = \frac{V_{ci}}{T_{ci}} \quad \text{Equation 6-47}$$

$$\beta = \frac{V_{ci}}{P_{ci}} \quad \text{Equation 6-48}$$

where T_{ci}, P_{ci}, V_{ci} are the available tensile, compressive, and in-plane shear strength per unit width for each half of SC panel section (kip/ft), respectively. Thus:

$$T_{ci} = 0.5[\min(F_y A_s, F_u A_{sn})] \quad \text{Equation 6-49}$$

$$P_{ci} = 0.8(0.5P_{no}) \text{ where } P_{no} \text{ is per Equation 6-25} \quad \text{Equation 6-50}$$

$$V_{ci} = 0.5(0.95\kappa F_y A_s) = 0.475\kappa F_y A_s \quad \text{Equation 6-51}$$

where κ is given in Equation 6-34.

6.7 Design for Impactive and Impulsive Loads

The design of SC walls for safety-related nuclear facilities needs to be checked for impactive loads (such as tornado-borne missiles, whipping pipes, aircraft missiles, or other internal and external missiles) and for impulsive loads (such as jet impingement loads, blast pressure, and compartment pressurization).

The effects for impactive and impulsive loads are considered in extreme environmental and abnormal load combinations concurrent with other loads (refer to Section 6.4.1).

Aircraft missiles are addressed in NEI 07-13 (Reference 10.1.12) and are outside the scope of this document.

Both the localized and global effects of impactive and impulsive loads are considered in the design.

6.7.1 Evaluation of the Global Response of Steel-Plate Composite Walls to Impactive and Impulsive Loads

6.7.1.1 Classification of Steel-Plate Composite Walls for Impactive and Impulsive Loads

The available strength of the SC wall for impactive and impulsive loads is governed by flexural yielding or out-of-plane shear failure.

Section N9.1.6b of AISC N690-18 classifies SC walls as flexure-controlled if the available strength for the limit state of flexural yielding is less than the available strength for the limit state of out-of-plane shear failure by at least 25 percent.

Otherwise, SC walls are classified as shear-controlled. That is for SC walls with available strength for the limit state of flexural yielding ≤ 0.75, the available strength for the limit state of out-of-plane shear failure, the SC wall is flexure-controlled. Otherwise, the SC wall is shear-controlled.

6.7.1.2 Dynamic Increase Factors on Material Strength for Analysis Involving Impactive and Impulsive Loads

Section N9.1.6a of AISC N690-18 permits dynamic increase factors to be applied to static material strength of steel and concrete for purposes of determining section strength. The dynamic increase factors are given in Table 6-3.

Table 6-3 Dynamic increase factors

Material	Dynamic Increase Factor*	
	Yield Strength	Ultimate Strength
Carbon steel plate	1.29	1.10
Stainless steel plate	1.18	1.00
Reinforcing steel Grade 40 Grade 60	 1.2 1.10	 1.05 1.05
Concrete compressive strength	-	1.25
Concrete shear strength	-	1.10

Note - * Dynamic increase factor value is limited to 1.0 for materials when the dynamic load factor associated with impactive or impulsive loading ≤ 1.2

6.7.1.3 Ductility Ratios

The effects of impactive and impulsive loads are permitted to be determined using inelastic analysis with limits on the ductility ratio demand, μ_{dd}, defined as follows:

$$\mu_{dd} = \frac{D_m}{D_y} \qquad \text{Equation 6-52}$$

where D_m is the maximum displacement from analysis, and D_y is the effective yield displacement established using the cross sectional effective flexural stiffness for analysis calculated using Equation 6-15.

Table 6-4 Ductility ratio demand

Description of Element	Ductility Ratio Demand μ_{dd}
Flexure-controlled SC walls	{{
Shear-controlled SC walls (yielding shear reinforcement spaced ≤ 0.5 section thickness)	
Shear-controlled SC walls (other configurations of yielding or non-yielding shear reinforcement)	
For axial compressive loads *	}}2(a),(c)

Note * Axial stiffness is calculated using the material elastic modulus and the model section thickness calibrated in accordance with Section 6.4.3

6.7.1.4 Response Determination

Response of SC walls subjected to impulsive loads is determined by one of the following methods:

1. The dynamic effects of the impulsive loads are considered by calculating a dynamic load factor. The resistance available for the impulsive load is at least equal to the peak of the impulsive load transient multiplied by the dynamic load factor, where the calculation of the dynamic load factor is based on the dynamic characteristics of the structure and impulsive load transient. System response is determined by either a nonlinear time history analysis or, for well-defined impulse functions, selected from established response charts, such as those in Biggs (Reference 10.1.14).
2. The dynamic effects of impulsive loads are considered by using impulse, momentum, and energy balance techniques. Strain energy capacity is limited by the ductility criteria in Section N9.1.6b of AISC N690-18, or the plate principal strain can be limited to 0.05.
3. The dynamic effects of impulsive loads are considered by performing a time history dynamic analysis. The mass and inertial properties are included as well as the nonlinear stiffness of the structural members under consideration. Simplified bilinear definitions of stiffness are acceptable. The maximum predicted response is governed by the ductility criteria in Section N9.1.6b of AISC N690-18.

In addition to the deformation limits shown in Table 6-4, the following analysis is performed for the critical impulsive/impactive load-wall combination, to check that the wall's intended function is not impaired:

- A mathematical model of the structural wall subjected to the impactive or impulsive loads is built. The model includes nearby structural members such as perpendicular walls and floor slabs.
- A time-history dynamic analysis is performed considering constitutive material models so that the nonlinear response of the structural members is obtained.
- Based on the internal forces and maximum displacements due to the computed response, the wall's intended function is then evaluated.

- The time history analysis also yield displacement/acceleration time histories throughout the structural model that are used to assess the safety-related function of nearby systems and components.

6.7.2 Evaluation of the Local Response of Steel-Plate Composite Walls to Impactive and Impulsive Loads

The required faceplate thickness of the SC wall is determined to prevent perforation of the SC wall by a missile.

The faceplate thickness required to prevent perforation is at least 25 percent greater than that calculated using rational methods.

Any rational method can be used to calculate the faceplate thickness required to prevent perforation under projectile impact. Section N9.6c of the Commentary of AISC N690-18 presents the following three-step approach to design an individual SC wall for a specific missile. This approach is based on a journal paper by Bruhl, et al. (Reference 10.1.13).

Using the method from Reference 10.1.13, the front surface faceplate is conservatively neglected. Thus, impact of a projectile (missile) on concrete dislodges a conical concrete plug, which, in turn, impacts the rear faceplate. This is illustrated in Figure 6-5. The evaluation and design steps are described in detail as follows.

Figure 6-5 Evaluation procedure against impact and conical plug geometry

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}}2(a),(c)

Step 1 - The design method involves first selecting a concrete wall thickness (concrete infill thickness) t_c using one of the following two methods:

1. Selecting wall thickness t_c based on other design requirements of the SC wall
2. 70 percent of the thickness for a RC wall determined using Section 6.3.2.1.2 of DOE-STD-3014-2006 (Reference 10.1.11). The 70 percent is an initial assumption for determining the required wall thickness based on Section N9.1.6c of the commentary of Reference 10.1.3.

$$t_c = 8.4\left(\frac{200}{V_o}\right)^{0.25}\left(\frac{MV_o^{\ 2}}{\frac{D}{12}f_c{}'*144000}\right)^{0.5} \qquad \text{Equation 6-53}$$

where t_c is in inches, V_o is the missile impact velocity (ft/sec),

M is the mass of missile = W_m/g, where W_m is the missile weight (lb), 8.4 reflects the 70 percent reduction multiplied by 12 to convert the thickness from feet to inches, g = 32.2 ft/sec^2, and

D is the average outer diameter of the missile (in.)

Step 2 - Next, the residual velocity of the missile after passing through concrete, V_r, is estimated using the formula in Section 2.1.2.4 of NEI 07-13 (Reference 10.1.12) as modified by Bruhl, et al. V_r is calculated per Equation 6-54. The ejected concrete plug is assumed to travel at the same residual velocity as both impact the rear faceplate.

$$V_r = \sqrt{\frac{1}{1 + \frac{W_{cp}}{W_m}}(V_o^2 - V_p^2)} \text{ For } V_o > V_p \qquad \text{Equation 6-54}$$

where W_{cp} represents the weight of the concrete plug (lb) ejected by the perforating missile weight W_m as given in Equation 6-55 (Refer to Figure 6-5)

$$W_{cp} = \pi\rho_c\left(\frac{t_c}{3}\right)(r_1^2 + r_1 r_2 + r_2^2) \qquad \text{Equation 6-55}$$

$$r_1 = \frac{D}{2} \qquad \text{Equation 6-56}$$

$$r_2 = r_1 + t_c(\tan\theta) \qquad \text{Equation 6-57}$$

$$\theta = \frac{45^\circ}{\left(\frac{t_c}{D}\right)^{1/3}} \qquad \text{Equation 6-58}$$

where ρ_c is the concrete unit weight in consistent units, so that W_{cp} is in lb.

The concrete wall perforation velocity V_p is calculated using the procedure described in Section 2.1.2.4 of NEI 07-13. Bruhl, et al. simplifies the procedure as follows:

1. The equivalent diameter of the missile, d_m, can be calculated from the missile contact area, A_{cm}. For a flat-nosed missile, the equivalent diameter equals the average outer diameter of the missile.

$$d_m = \sqrt{\frac{4A_{cm}}{\pi}} \qquad \text{Equation 6-59}$$

2. The strength coefficient is calculated as

$$k = \frac{180}{\sqrt{f_c'(psi)}} \qquad \text{Equation 6-60}$$

3. Determine missile shape factor using α_p = 0.6 for deformable missiles and α_p = 1.0 for rigid missiles, as follows:
   a. N = 0.72 for flat-nosed missiles
   b. N = 0.84 for blunt-nosed missiles
   c. N = 1.0 for bullet-nosed missile
   d. N = 1.14 for sharp-nosed missile
   e. N = 1.14 In Equation 6-62 if the missile diameter ≤ 5.9 in
   f. N is evaluated using Equation 6-61 for hollow missiles (such as pipes)

$$N = 0.72 + \left(\left(\frac{D}{d_m}\right)^2 - 1\right)0.0306 \le 1.0 \qquad \text{Equation 6-61}$$

4. If $(t_c/\alpha_p d_m) \le 2.65$

$$V_p = 1000 d_m\left[\frac{d_m}{1.44kW_m N\beta^2}\left(2.2 - \sqrt{4.84 - 1.2\left(\frac{t_c}{d_m ap}\right)}\right)^2\right]^{5/9} \qquad \text{Equation 6-62}$$

5. If $2.65 < (t_c/\alpha_p d_m) < 3.27$

$$V_p = 1000 d_m\left[\frac{d_m}{4kWN\beta^2}\left(\frac{t_c}{1.29\alpha_p d_m} - 0.53\right)^2\right]^{5/9} \qquad \text{Equation 6-63}$$

6. If $(t_c/\alpha_p d_m) \ge 3.27$

$$V_p = 1000 d_m\left[\frac{1}{kW_m N\beta}\left(\frac{t_c}{1.29\alpha_p} - d_m(0.53 + \beta)\right)\right]^{5/9} \qquad \text{Equation 6-64}$$

where d_m and t_c are in inches, and W_m is in lb.

The factor β = 1.45 is recommended for use in Bruhl, et.al. (Reference 10.1.13). (More information is provided in the reference for the selection of the value of β.)

Step 3 - The required faceplate thickness, t_p, using the formula from Reference 10.1.13 is as follows:

1. Calculate m

$$m = \frac{W_m + W_{cp}}{386\ in/sec^2} \qquad \text{Equation 6-65}$$

2. Calculate the quasi-static radial compressive true stress δ_s (psi)

$$\delta_s = 5.1F_y + 101000, \text{ for } t_p \geq 0.25 \text{ in} \qquad \text{Equation 6-66}$$

where F_y is the faceplate yield strength in psi.

3. The faceplate thickness (in.) required to prevent perforation due to impact is calculated as

$$t_p^r = 1.25\left[0.72\left(\frac{(12V_r)^2 m}{\frac{\pi}{2}d_m^2\ \sigma_s}\right)\right] \qquad \text{Equation 6-67}$$

where the factor 12 in Equation 6-67 is to convert V_r from ft/sec to in./sec and the factor 1.25 is to satisfy the requirement of Section N9.1.6c of AISC N690-18 for the thickness of the faceplate to be 25 percent greater than the thickness calculated using rational methods.

6.8 Design and Detailing Around Openings

The load redistribution around an opening creates stress concentrations whose severity depends on factors such as size of the opening, presence or absence of sharp re-entrant corners, and type and magnitude of loading.

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 is manifested some distance away from the opening.

Openings are categorized as small if the largest opening dimension ≤ $t_{sc}/2$ and as large if that dimension > $t_{sc}/2$.

6.8.1 Design and Detailing Around Small Openings

The following provisions apply for the design and detailing of a small opening with a free edge at the opening perimeter.

1. Analysis can be performed without modeling the opening.
2. The SC panel section where the opening is located is evaluated considering a 25 percent reduction in available strengths. If one panel section encompasses the opening (Opening A in Figure 6-6), the strength of just that panel section is reduced. If the opening lies in more than one panel section (Opening B in Figure 6-6), the strength of panel sections that partially include the opening is reduced by 25 percent.

Figure 6-6 Evaluation procedure against impact and conical plug geometry

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}}2(a),(c)

3. Re-entrant corners of noncircular or non-oval openings have radii $r \geq 4t_p$
4. The first row of ties around the opening is located at a distance $\leq t_{sc}/4$

For design and detailing for a fully developed edge at the opening perimeter, sections surrounding the opening are permitted to be designed using the design demands based on an analysis model that does not consider the opening, provided the following detailing requirements are satisfied.

1. Reentrant corners of noncircular or non-oval openings have corner radii $r \geq 4t_p$.
2. A steel sleeve is provided to span across the openings to the opposite faceplates.
   a. Nominal yield strength of the sleeve is ≥ nominal yield strength of the faceplate.
   b. And $t_{sleeve} \geq t_p$.
3. The steel sleeve is anchored into the surrounding concrete in accordance with the requirements of Section N9.1.3 of AISC N690-18, where the width-to-thickness ratio is calculated using the sleeve thickness instead of the faceplate thickness.
4. A welded flange is fitted at each end of the sleeve made from plate material with:
   a. $F_{y\text{-}flange} \geq F_{y\text{-}faceplate}$
   b. $t_{flange} \geq t_p$

c. The flange extends to a distance of at least the section thickness beyond the opening perimeter.

d. The flange is connected to the sleeve using complete-joint-penetration (CJP) groove welds.

e. The flange is joined with the surrounding faceplate in one of the following ways:

   i) If the flange is less than 25 percent thicker than the surrounding faceplate, the faceplate is joined with the sleeve using a CJP groove weld and the flange joined with the faceplate using the maximum size fillet weld permitted by the specification. Refer to Figure 6-7.

   ii) If the flange is greater than or equal to 25 percent thicker than the faceplate, the faceplate is joined with the flange only along its outer perimeter with a CJP groove weld. Refer to Figure 6-8.

Figure 6-7 Small circular opening- fully developed edge, $t_{flange} < 1.25t_p$

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}}2(a),(c)

Figure 6-8 Small circular opening - fully developed edge, $t_{flange} \geq 1.25\ t_p$

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}}2(a),(c)

6.8.2 Design and Detailing Around Large Openings

At the boundary of large openings, detailing is provided to achieve either a free edge or a fully-developed SC wall. Design and detailing is accomplished as follows:

The following provisions apply to the design and detailing for a free edge at the opening perimeter.

1. The size of the opening modeled for analysis purposes is larger than the physical opening such that it extends to where the faceplates are fully developed away from the boundary of the opening (Figure 6-9). Refer to Equation 6-6 for calculating the development length L_d. Development length is limited to $L_d \leq 3t_{sc}$.

Figure 6-9 Modeling of large opening with free edge at opening perimeter

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}}[2(a),(c)]

2. No reductions are applied to the available strengths of the panel sections in the vicinity of the as-modeled opening.
3. Re-entrant corners of noncircular or non-oval openings have corner radii ≥ $4t_p$.
4. The first row of ties around the opening is located at a distance ≤ $t_{sc}/4$

For design and detailing for a fully-developed edge at the opening perimeter, fully-developed SC walls around large openings are modeled and designed considering the physical boundary of the opening and follow the provisions for fully-developed small openings.

6.8.3 Design and Detailing of Bank of Small Openings

The region affected by a concentrated bank of small openings is considered as a large opening when the clear distance between adjacent small openings is ≤ $2t_{sc}$ for openings designed and detailed for the free edge at the opening perimeter and ≤ t_{sc} for openings designed and detailed for the fully-developed edge at the opening perimeter.

The physical dimension of the large opening is equal to the distance between the outermost edges of the bank of small openings. Dimensions of the as-modeled large opening is discussed in Section N9.1.7b of AISC N690-18 and Section 6.8.2.

6.9 Ductile Failure of Faceplates with Holes

Per Section N9.1.1 of AISC N690-18, for faceplates with holes, the effective rupture strength per unit width, $F_u A_{sn}$, is greater than the yield strength per unit width, $F_y A_s$, Refer to Section 6.6.1 and Equation 6-24.

6.10 Requirements for Steel Rib Embedment

Steel ribs can be welded to the faceplates of SC walls to increase the stiffness and strength of the empty modules, improve the resistance of the faceplates to hydrostatic pressure from concrete casting, and prevent local buckling of the faceplates after concrete hardening.

Per Section N9.1.1 of AISC N690-18, steel ribs are embedded into the concrete no more than the lesser of six in. or the embedment depth of the steel anchor minus two in. as shown in Figure 6-10. The ribs are welded to the faceplates and anchored in the concrete to develop 100 percent of their nominal yield strength. The contribution of steel ribs, including composite action and strength, is not considered for any design parameters.

Figure 6-10 Embedment depth of steel ribs

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}}[2(a),(c)]

6.11 Splices Connection Strength Requirement

Per Section N9.1.1 of AISC N690-18, splices between faceplates are to be welded using CJP groove welds, or bolted to develop the nominal yield strength of the two (spliced) faceplates.

7.0 Design Methodology for Steel-Plate Composite Walls Connections

This section presents a design methodology for the SC wall connection to be used in the design and construction of the RXB, the CRB, and the RWB. Design of the SC walls is presented in previous Section 6.0. The design methodology for SC wall connection is in compliance with the requirements of ANSI/AISC N690-18 (Reference 10.1.3) and ANSI/AISC 360-16 (Reference 10.1.15).

7.1 General Requirements for Steel-Plate Composite Walls Connections

The requirements for designing SC walls are summarized in the subsections below. Steel-plate composite walls of safety-related nuclear facilities are connected to each other and anchored to the concrete basemat.

The following are the types of connections between SC wall structures and their adjoining structures in accordance with Section N9.4 of AISC N690-18:

- Splices between SC wall sections
- Splices between SC walls and RC walls
- Connections at the intersection of SC walls
- Connections at the intersection of SC walls and RC walls
- Connections between SC walls and RC basemat
- Connections between SC walls and RC slab

A connection may be defined as the assembly of steel connectors. Connectors consist of steel-headed stud anchors, anchor rods, tie bars, reinforcing bars and dowels, post-tensioning bars, shear lugs, embedded steel shapes, welds and bolts, rebar mechanical couplers, and direct bearing in compression and the surrounding concrete materials anchoring the rebar or providing bearing resistance that participate in the force transfer mechanisms for tension, compression, in-plane shear, out-of-plane shear, and out-of-plane flexure between two connected parts.

Steel connectors do not include any portions of the SC walls being connected. The connection region is specifically designed to undergo ductile yielding and energy dissipation during overloads. Force transfer from the composite SC wall to the supports or connected structures occurs within these connection regions.

For design purposes, SC walls are divided into interior regions and connection regions. Additionally, the connection regions serve as transition regions wherein the faceplates and concrete infill of SC walls redistribute forces according to their relative stiffness and develop composite action.

For design purposes, SC walls are divided into interior regions and connection regions in accordance with Section N9.2 of AISC N690-18.

Connection regions consist of perimeter strips with a length not less than the SC wall thickness, t_{sc}, and not more than twice the SC wall thickness, $2t_{sc}$. Figure 6-2 illustrates the typical connection regions for SC walls. The requirement for connection region length is based on typical development lengths of No. 11 to No. 18 reinforcing bars used typically in nuclear construction. Specifying connection region length less than the wall thickness ($\leq t_{sc}$c) can be impractical and lead to detrimental congestion of steel anchors and ties.

Wall-to-wall connections, wall anchorages, and wall splice connections are rigid for out-of-plane moment transfer. Wall-to-slab connections are rigid or pinned, consistent with the analysis model used.

If more than one force mechanism is possible for transferring force in the connection, the one providing the greatest strength for that demand type is assumed to transfer the forces.

7.2 Design of Steel-Plate Composite Wall Connection

7.2.1 Required Connection Strength

Section N9.4.2 of AISC N690-18 allows the design of SC walls based on one of two philosophies:

- Full-strength connection: This connection is designed to be stronger than the expected strength of the weaker of the two connected parts. For this case, the required strength of the full-strength connection is determined as 125 percent of the smaller of the corresponding strength of the connected parts.

  This design philosophy ensures ductile behavior with yielding and inelasticity occurring away from the connection in one of the connected parts. It is preferred for use in connection design where feasible.

- Over-strength connection: This connection design is only used in limited situations when the full-strength design is not feasible, and limited for use only for the particular force transfer mechanisms in the connection that cannot be designed to achieve the full strength of the connected wall. An example is designing a connection to an SC wall or RC slab that is overdesigned with respect to demand because of radiation shielding requirements. Over-strength connections are expected to fail before the connected SC walls; therefore, these connections are checked for the simultaneous actions of amplified design demands calculated from an FEA of loading combinations. That means that the required strength of the over-strength connection is determined as 200 percent of the required strength because of seismic loading plus 100 percent of the required strength because of nonseismic loads, including thermal loads.

  The connectors utilized in over-strength connection force transfer mechanisms are designed to exhibit ductile failure modes involving steel yielding.

7.2.2 Required Connection Demand

Connection demands per unit length of the connection are obtained from an FEA conducted in accordance with Section N9.2 of AISC N690-18. Elastic FEAs are conducted for static loading conditions, and for Condition A (operating thermal + seismic loading) and Condition B (accident thermal + seismic loading), using appropriate stiffness values to account for the effects of concrete cracking where applicable.

The results from the FEAs are used to determine the design demands per unit length of the connection, depending on the connection design philosophy. The demands are membrane axial force, N_u, membrane in-plane shear force, V_u^{in}, out-of-plane shear force, V_u^{out}, and out-of-plane bending moment, M_u. These are illustrated in Figure 11-1 from AISC Steel Design Guide 32 (Reference 10.1.5).

Based on the connection design philosophy, each connection design demand can be differentiated into demands due to seismic loading (Condition A or B) and demands due to nonseismic loading, such as static loads and thermal loads.

7.3 Connection Detailing

This section describes typical connection configurations for SC walls: (a) SC wall-to-basemat anchorage, (b) SC wall-to-SC wall connection, and (c) SC wall-to-RC slab connection. It describes the detailing and possible force transfer mechanisms based on the Commentary in AISC N690-18.

7.4 Steel-Plate Composite Wall-to-Basemat Anchorage

The SC wall-to-basemat connections can be detailed differently depending on project-specific design considerations and site conditions. The connections can be designed as full-strength or over-strength connections.

Three typical configurations of the SC wall-to-basemat connections are discussed here. Each connection configuration is discussed along with possible force transfer mechanisms. The adequacy of the connection, considering the connection is designed as a full-strength connection, is checked for individual demand types corresponding to 1.25 times the available strength of the SC wall. The connection is also checked for a combination of demands, obtained from an FEA, in the wall for different load combinations.

7.4.1 Single Base Plate Connection

The single base plate SC wall-to-basemat connection consists of a base plate welded to the faceplates of the SC wall. The base plate is connected to the concrete infill of SC wall by steel anchors and to the basemat by welded coupled bars. The connection layout and typical connection detailing is presented in Figure 11-4 from AISC Steel Design Guide 32 (Reference 10.1.5).

The force transfer mechanism for individual demand types is discussed below. The section discusses only one force transfer mechanism for each demand type; however, more than one force transfer mechanism is possible for some demand types, in which case the one with the largest connector design strength is the governing mechanism.

Tensile Force Demand: The SC wall-to-base plate weld is designed for 1.25 times the available tensile strength of the SC wall. The mechanism conservatively ignores force transfer through the steel anchors at the SC concrete infill base plate interface. The force in the base plate transfers to the concrete basemat by anchor rods welded to the base plate and embedded in the basemat concrete as shown in Figure 11-5 from Steel Design Guide 32.

Compression Force Demand: The compression force is transferred through bearing to the base plate and the base plate transfers it to the basemat. The limit states of bearing for the concrete and yielding for the base plate are checked for this force transfer mechanism. Cantilever bending in the base plate due to the reaction from basemat is also considered (refer to Figure 11-6 from Steel Design Guide 32).

In-Plane Shear Demand: The in-plane shear strength demand in the SC wall is transferred to the base plate by the steel anchors. The fraction of in-plane shear demand carried by the faceplates is transferred to the base plate through the connection between the faceplate and the baseplate (typically a welded connection). The demand is then transferred to the basemat by means of shear friction force between the base plate and the basemat (refer to Figure 11-7 from Steel Design Guide 32).

Other force transfer mechanisms also can be considered for transferring force from the base plate to the basemat, for example, shear lugs or concrete bearing on rebar couplers.

Out-of-Plane Shear Demand: The out-of-plane shear strength is governed by the available shear strength of the steel anchors, and the friction force between the base plate and the basemat concrete (refer to Figure 11-8 from Steel Design Guide 32).

Out-of-Plane Flexural Demand: The out-of-plane flexural demand can be considered as an equivalent force couple acting on the faceplates. The resulting tension and compression forces in the faceplates are transferred to the basemat as discussed earlier in this section for tensile and compression demand transfer and shown in Figure 11-9 from Steel Design Guide 32.

7.4.2 Split Base Plate Connection

The split base plate SC wall-to-basemat connection is a variant of the single base plate connection, where two separate base plates are provided instead of a single base plate, and no steel anchors are provided because there is not enough interface between the base plates and the SC concrete infill is available.

Typical layout of the connection is presented in Figure 11-10 from Steel Design Guide 32. The force transfer mechanism for individual demand types is discussed below.

Tension and Out-of-Plane Flexural Demands: Tension and out-of-plane demands are transferred in the same mechanism as in the single base plate connections discussed in Section 7.4.1.

Compression Force Demand: The compression force in the SC wall concrete infill is transferred in bearing directly to the basemat concrete.

In-Plane and Out-of-Plane Shear Demands: In-plane and out-of-plane shear demands are transferred through shear friction at the interface of the SC concrete infill and the basemat concrete. Other force transfer mechanisms, such as shear transferred through the base plate and anchoring rebar, can be considered.

7.4.3 Rebar Steel-Plate Composite Wall-to-Basemat Connection

For this type of connection, reinforcement bars are used to transfer SC wall demands to the basemat. The details of the connection can be varied depending on whether the connection is full strength or over-strength and constructability concerns.

An over-strength connection can be achieved by leaving reinforcing bars projecting from the concrete basemat where an empty SC module is assembled around the reinforcing bars followed by casting of the concrete, or the concrete is cast monolithically with the basemat.

The faceplates can be stopped at the basemat surface or embedded in the basemat. The option of embedding the faceplates in the basemat is hard to construct and can interfere with basemat reinforcement. This also requires an increase in the embedment depth required for the faceplates if the faceplates are used as force transfer mechanism. The connection detail with faceplates embedded is presented in Figure 11-11 from Steel Design Guide 32.

The second option of ending the faceplates at the surface base plates is easier to construct; however, in this case, the bottom portion of the SC wall behaves as an RC wall. This type of connection can be used for over-strength design, because achieving the full strength of the SC wall through a connection that essentially behaves as an RC wall-to-basemat connection can result in heavy reinforcement. The amount of reinforcement can adversely affect the constructability of the connection.

The force transfer mechanism for individual demand types is discussed below.

Tensile Demand: The tensile demand in the SC wall is transferred from the faceplates to the concrete infill in the SC-to-RC transition region through the steel-headed anchors. The forces are then transferred to the reinforcing rebars through bond. Refer to Figure 11-12 from Steel Design Guide 32.

The amount of reinforcement is determined based on the tensile demand. Sufficient embedment length of reinforcement bars [lap splice length per ACI 349-13, (Reference 10.1.4)] is provided to develop the tensile strength of the bars.

Compression Force Demand: The compression force demand in the SC wall is transferred to the basemat in bearing. The limit state of bearing for concrete is checked for this force transfer mechanism.

In-Plane and Out-of-Plane Shear Demand: The force transfer mechanism for in-plane and out-of-plane shear are similar. The shear forces in the faceplates are transferred to the concrete infill through the faceplate steel-headed stud anchors. The force from the concrete infill transfers through shear friction between the infill concrete and the basemat concrete. Direct shear in the rebar-shear force transferred from the SC wall directly to the basemat rebar-also is checked in order to avoid failure of the connection. Refer to detail Figure 11-13 from Steel Design Guide 32.

Out-of-Plane Flexural Demand: The out-of-plane available flexural strength of the connection can be idealized as a plastic compression mechanism in the basemat concrete, where compression in concrete reaches its capacity, and tension forces is applied to the reinforcement bars on the opposite side. The force transfer mechanism is shown in detail in Figure 11-14 from Steel Design Guide 32. Adequate resistance and development length of the rebar need to be provided to transfer the force demands from the SC walls to the concrete basemat.

7.5 Steel-Plate Composite Wall to Steel-Plate Composite Wall Joint Connection

This type of connection is designed as a full-strength or over-strength connection. The typical SC wall-to-SC wall T-joint connection consists of a discontinuous SC wall that is connected to an orthogonal continuous SC wall refer to Figure 11-15 from Steel Design Guide 32. The force demands are transferred from the discontinuous SC wall to the continuous SC wall through the joint region.

For a full-strength connection, the individual demand types are transferred by considering the following force transfer mechanisms.

Axial Tensile Demand: Full-strength connection design corresponds to the development of a plastic mechanism. This is accompanied by the formation of plastic hinges in the continuous SC wall at locations outside the joint region as shown in Figure 11-16 from Steel Design Guide 32.

This plastic mechanism provides ductility and energy dissipation for beyond design basis events and limits the maximum axial tension for which the joint is designed.

If $M_p^{exp\text{-}c}$ is the expected plastic flexural strength of the continuous SC wall, the maximum axial tension, N_r, that can be transferred through the connection is given by

$$N_r = 4\frac{M_p^{exp-c}}{L} \qquad \text{Equation 7-1}$$

where L is the clear span of the wall between the supports, refer to Figure 11-16 from Reference 10.1.5.

The joint region is to be designed for the forces and joint shears, V_{js}, associated with connection required strength, N_r, in axial tension. Refer to Figure 11-17 from Steel Design Guide 32.

The force transfer mechanism due to axial tensile strengths is shown in Figure 11-17 from Steel Design Guide 32. The various connector elements need to be designed for the demands corresponding to the force transfer mechanism.

In-Plane Shear Demand: Full-strength connection design requires transfer of expected in-plane shear strength, $V_n^{i\text{-}exp\text{-}dc}$, which is the design in-plane shear demand of the discontinuous SC wall-to-the continuous SC wall. The various connector elements in the joint region are designed for the forces and joint shear demands associated with the mechanism shown in Figure 11-18 from Steel Design Guide 32.

Out-of-Plane Shear Demand: Full-strength connection design requires transfer of the expected out-of-plane shear strength, $V_n^{o\text{-}exp\text{-}dc}$, which is the design out-of-plane shear demand, for the connection of the discontinuous SC wall as shown in Figure 11-19 from Steel Design Guide 32. The various connector elements in the joint region are designed for the forces and joint shear, V_{js}, demands associated with the mechanism shown in Figure 11-20 from Steel Design Guide 32.

Out-of-Plane Flexural Demand: Full-strength connection design requires transfer of the expected out-of-plane flexural strength, $M_p^{exp\text{-}dc}$, which is the design out-of-plane flexure demand of the discontinuous SC wall. The mechanism corresponds to the development of a plastic hinge in the discontinuous SC wall as shown in Figure 11-21 from Steel Design Guide 32. This plastic hinge provides ductility and energy dissipation for beyond design basis events. The various connector types in the joint region are designed for the forces and joint shear demands associated with the transfer mechanism shown in Figure 11-22 from Steel Design Guide 32.

Joint Shear Demand: The joint region for SC wall-to-wall connection is designed to be stronger than the connected walls. The design consists of steel diaphragm plates, steel anchors, and tie bars. The steel diaphragm plates confine the concrete and the region is treated as a filled composite column. The joint region is designed for, V_{js}, associated with the transfer of each demand type as follows:

- The joint shear V_{js} due to tensile demand transfer mechanism.
- The joint shear V_{js} due to out-of-plane shear demand transfer mechanism, refer to Figure 11-20 from Steel Design Guide 32. This contributes to the joint shear.

- The joint shear V_{js} due to out-of-plane flexural demand transfer mechanism. Refer to Figure 11-22 from Steel Design Guide 32. This contributes to the joint shear.

Based on ACI 349-13, the joint shear strength for an SC wall-to-wall joint is:

$$V_n = 12\sqrt{f_c'}A_j \qquad \text{Equation 7-2}$$

Where A_j is the area of the joint calculated as the width times the depth of the joint.

7.6 Reinforced Concrete Slab to Steel-Plate Composite Wall Joint

Figure 11-23 from Steel Design Guide 32 presents a typical RC slab-to-SC wall connection detail. The connection can be designed as a full-strength or over-strength connection. The connection consists of a discontinuous RC slab connected to an orthogonal continuous SC wall at both sides. The force demands need to be transferred from the discontinuous RC slab-to-the continuous SC wall through the joint region.

For a full-strength connection, the individual demand types can be transferred by considering the following force transfer mechanisms:

Axial Tensile Demand: Axial tension is transferred from the RC slab to the SC wall through a load path consisting of rebar, rib plates, or ties, which are in direct tension. The tensile demand type is shown in detail Figure 11-25 from Steel Design Guide 32. The load transfer connectors are developed adequately to transfer the factored demand.

In-Plane and Out-of-Plane Shear Demand: Steel anchors welded to the outside of the SC wall embedded in the RC slab and the anchors located on the inside of the SC wall provide the direct load path for the in-plane and out-of-plane shear demands. Detail Figure 11-26 and Figure 11-27 from Steel Design Guide 32 show the in-plate and out-of-plane shear demand for the connection. Shear friction resulting from engaging slab rebar at the SC faceplate provides an additional, independent, full-strength force transfer mechanism.

Out-of-Plane Flexure Demand: Figure 11-28 from Steel Design Guide 32 shows the out-of-plane flexural demand for the connection. This demand is transferred from the slab to the SC wall through direct tension in rebar, rib plates, or ties. The resulting compression force is transferred mainly through concrete in bearing.

The out-of-plane flexural demand leads to a joint shear demand for the connection, as shown in detail Figure 11-28 from Steel Design Guide 32. The demand resisted by the concrete joint shear capacity is determined using ACI 349-13.

7.7 Connections Available Strength

The available strength of the connections is determined based on requirements of AISC N690-18. The applicable force transfer mechanism and the available strength of the connectors contributing to the force transfer mechanism is considered to evaluate the available strength. Note that some connections can be governed by more than one code. These connections meet the requirements of applicable codes. Sections below provide general guidance for designing the connector elements from their respective standards as given in AISC N690-18.

7.7.1 Steel-Headed Anchors

The available strength for steel anchors is determined in accordance with AISC 360-16 as described in the sections below. This section applies to cast-in-place headed stud anchors only. Information about different types of anchors can be obtained from the sections referenced below.

7.7.2 Shear Strength of the Steel-Headed Anchors

The shear strength of the steel-headed anchors are calculated as:

1. Where concrete breakout strength in shear is not an applicable limit state, the design shear strength, Q_{cv}, of one steel-headed stud anchor is as given in Equation 7-3 below.

$$Q_{cv} = 0.65 F_{ua} A_{sa} \qquad \text{Equation 7-3}$$

   which accounts for the strength reduction factor.

2. Where concrete breakout strength in shear is an applicable limit state, the available shear strength of one steel-headed stud anchor is determined as follows:

   a. Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318-08, (Reference 10.1.16) on both sides of the concrete breakout surface for the steel-headed stud anchor, the minimum of the steel nominal shear strength and the nominal strength of the anchor reinforcement is used for the nominal shear strength. Thus, the available design shear strength is calculated as:

$$Q_{cv} = 0.65 \times \min(F_{ua} A_{sa}, F_{ur} A_{sr}) \qquad \text{Equation 7-4}$$

   b. Otherwise, the shear strength of the anchor stud is calculated using Appendix D of ACI 318-08, as explained below. The applicable strength reduction factor, ϕ_V is taken from Appendix D of ACI 318-08 for Condition B considering no supplementary reinforcement is provided. Refer to Appendix D ACI 318-08 otherwise.

   Per Appendix D of ACI 318-08, the nominal strength of steel-headed anchor in shear, when not governed by steel failure of anchors, is the minimum of concrete breakout strength in shear and concrete pryout strength in shear.

$$Q_{cv} = \text{minimum}(\phi_v V_{sa},\ \phi_v V_{cbg} \text{ or } \phi_v V_{cp} \text{ or } \phi_v V_{cpg})$$

i) $\phi_v V_{sa}$, steel strength of the anchors in shear using Appendix D of ACI 318-08.

$$\phi_v V_{sa} = 0.65 n_a A_{se} f_{uta} \quad \text{Equation 7-5}$$

f_{uta} is taken as the max ($1.9f_{ya}$, 125ksi)

Per Appendix D of ACI 318-08, where anchors are used with built-up grout pads, the shear strength from Equation 7-5 is multiplied by a 0.80 factor.

ii) $\phi_v V_{cb}$ or $\phi_v V_{cbg}$, concrete breakout strength of anchor in shear using Appendix D of ACI 318-08, where the available concrete breakout strength, $\phi_v V_{cb}$ or $\phi_v V_{cb}$ is given as:

1. Concrete breakout strength for a single anchor, $\phi_v V_{cb}$, with shear force perpendicular to the edge on a single anchor

$$\phi_v V_{cb} = 0.7 \frac{A_{vc}}{A_{vco}} \Psi_{c,V} \Psi_{ed,V} \Psi_{h,V} V_b \quad \text{Equation 7-6}$$

2. Concrete breakout strength for a group of anchors, $\phi_v V_{cbg}$, for shear force perpendicular to the edge on a group of anchors

$$\phi_v V_{cbg} = 0.7 \frac{A_{vc}}{A_{vco}} \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} V_b \quad \text{Equation 7-7}$$

3. For shear force parallel to an edge, $\phi_v V_{cb}$ or $\phi_v V_{cbg}$ is permitted to be twice the value of the shear force determined from Equation 7-6 or Equation 7-7, respectively, with the shear force assumed to act perpendicular to the edge and with $\Psi_{ed,V}$ taken equal to 1.0.

4. For anchors located at a corner, the limiting nominal concrete breakout strength is determined for each edge, and the minimum value is used.

   A_{Vco} is the projected area for a single anchor in a deep member with a distance from edges equal or greater than $1.5c_{a1}$ in the direction perpendicular to the shear force. It is permitted to evaluate A_{Vco} as the base of a half-pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$. Refer to D.6.2.1 of ACI 318-08.

$$A_{Vco} = 4.5(c_{a1})^2 \quad \text{Equation 7-8}$$

   c_{a1} is the distance from the center of an anchor shaft to the edge of concrete in the direction of the applied shear. In the case of anchors located at varying distances from the edge, refer to Appendix D of ACI 318-08 for additional direction.

Where anchors are located in narrow sections of limited thickness such that both edge distances c_{a2} and thickness are less than 1.5c_{a1}, the value of c_{a1} used for the calculation of the projected area does not exceed the largest of:

a. c_{a2}/1.5, where c_{a2} is the largest edge distance,

b. h_a/1.5, and

c. s/3, where s is the maximum spacing perpendicular to direction of shear, between anchors within a group.

A_{Vc} is the projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. It is permitted to evaluate A_{Vc} as the base of a truncated half pyramid projected on the side face of the member where the top of the half pyramid is given by the axis of the anchor row selected as critical. A_{Vc} does not exceed nA_{Vco}.

V_b is the basic concrete breakout strength in shear of a single anchor in cracked concrete as given below; correction factors for lightweight concrete are not included because use of lightweight concrete is prohibited for SC walls construction.

$$V_b = \min\left(7\left(\frac{I_e}{d_s}\right)^{0.2}\sqrt{d_s}\sqrt{f_c'}(c_{a1})^{1.5}, 9\sqrt{f_c'}(c_{a1})^{1.5}\right) \qquad \text{Equation 7-9}$$

$$I_e = \max(h_{ef}, 8d_s) \qquad \text{Equation 7-10}$$

The modification factor for anchor groups loaded eccentrically in shear is given in Equation 7-11, where e_v' is the distance between resultant shear load on a group of anchors loaded in shear (only anchors that are loaded in shear in the same direction), and the centroid of the group of anchors loaded in shear in the same direction:

$$\Psi_{ec,V} = \frac{1}{\left(1 + \frac{2e_v'}{2c_{a1}}\right)} \le 1 \qquad \text{Equation 7-11}$$

The modification factor for edge effect for a single anchor or group of anchors loaded in shear is given in Equation 7-12 or Equation 7-13 based on c_{a2}, where c_{a2} is the distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}.

$$\Psi_{ed,V} = 1.0 \text{ if } c_{a2} \geq 1.5c_{a1}$$ Equation 7-12

$$\Psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \text{ if } c_{a2} \leq 1.5c_{a1}$$ Equation 7-13

For modification factor $\Psi_{c,V}$=1.4 is used for anchors located in a region assuming no cracking at service load. Refer to Appendix D of ACI 318-08 if other cases are of an interest in the analysis. The modification factor for anchors located in a concrete member where h_a < $1.5c_{a1}$, $\Psi_{h,V}$ is computed as

$$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$$ Equation 7-14

iii) $\phi_v V_{cb}$ or $\phi_v V_{cpg}$, concrete pryout strength of anchor in shear using Appendix D of ACI 318-08

1. For a single anchor

$$\phi_v V_{cp} = 0.7k_{cp}N_{cb}$$ Equation 7-15

2. For a group of anchors

$$\phi_v V_{cp} = 0.7k_{cp}N_{cbg}$$ Equation 7-16

k_{cp} = 1.0 for h_{ef} < 2.5 in.

k_{cp} = 2.0 for h_{ef} ≥ 2.5 in

N_{cb} and N_{cbg} are given by Equation 7-20 and Equation 7-21, respectively.

7.7.3 Tensile Strength of Steel-Headed Stud Anchors

The tensile strength of steel-headed anchors is calculated per ACI 318-08 Section I8.3b as follows:

1. If $c_{a1} \geq 1.5h_{an}$ and $S_{an} \geq 3h_{an}$, refer to Figure 7-1 the available tensile strength of one steel-headed stud is per Equation 7-17. Where c_{a1} is the distance from the center of the anchor to the free edge and h_{an} is the height of the steel-headed stud anchor measured to the top of the stud head.

$$Q_{ct} = 0.75F_{ua}A_{sa}$$ Equation 7-17

2. If c_{a1}<1.5h_{an} or if S_{an}<3h_{an} then
   a. Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318-08 on both sides of the concrete breakout surface for the steel-headed stud anchor,

   $$Q_{ct} = 0.75 \times \min(F_{ua}A_{sa}\ , F_{ur}A_{sr})$$ Equation 7-18

   b. Otherwise, the available strength of anchors in tension is to be calculated per Appendix D of ACI 318-08, as shown below. The value of ϕ factors in this section are based on Condition B, where no supplementary reinforcement is present. Otherwise, refer to Appendix D of ACI 318-08.

   $$Q_{ct} = \text{minimum}(\phi_t N_{sa}, \phi_t N_{cb} \text{ or } \phi_t N_{cbg}, \phi_t N_{pn} \text{ or } \phi_t N_{pn}, \phi_t N_{pn} \text{ or } \phi_t N_{sbg})$$

   i) $\phi_t N_{sa}$ is the tensile strength of anchor in tension using Appendix D of ACI 318-08.

   $$\phi_t N_{sa} = 0.75 n_a A_{se} f_{uta}$$ Equation 7-19

   ii) $\phi_t N_{cb}$ or $\phi_t N_{cbg}$ is the available concrete breakout strength in tension for a single anchor and a group of anchors respectively calculated per Appendix D of ACI 318-08.

   For a single anchor:

   $$\phi_t N_{cb} = 0.70\frac{A_{Nc}}{A_{Nco}}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$ Equation 7-20

   For a group anchors:

   $$\phi_t N_{cbg} = 0.70\frac{A_{Nc}}{A_{Nco}}\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$$ Equation 7-21

   Where A_{Nco} is the projected concrete failure area of a single anchor with an edge distance equal to or greater than 1.5h_{ef}:

   $$A_{Nco} = 9h_{ef}^2$$ Equation 7-22

   Where anchors are located less than 1.5h_{ef} from three or more edges, the value of h_{ef} used in this section is the greater of $c_{a,max}$ /1.5 and s/3 of the maximum spacing between anchors within the group.

A_{Nc} is the projected concrete failure area of a single or a group of anchors that is approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. $A_{Nc} \leq nA_{Nco}$, where n is the number of tensioned anchors in the group. Refer to Figure RD5.2.1 in ACI 318-08 for information regarding the calculation of the projected areas.

N_b is the basic concrete breakout strength of a single anchor in tension in cracked concrete:

$$N_b = 24\sqrt{f_c'}h_{ef}^{1.5} \quad \text{Equation 7-23}$$

The modification factors for anchor group loaded eccentrically in tension $\Psi_{ec,N}$ is calculated as:

$$\Psi_{ec,N} = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)} \leq 1.0 \quad \text{Equation 7-24}$$

Where e'_N is the eccentricity calculated as follows:

1. If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension are considered when determining the eccentricity e'_N for use in Equation 7-24.
2. In the case where eccentric loading exists about two axes, the modification factor $\Psi_{ec,N}$ is computed for each axis individually and the product of these factors used as $\Psi_{ec,N}$ in Equation 7-21.

   The modification factor for edge effects for a single anchor or anchor groups loaded in tensions is given below, where $c_{a,min}$ is the minimum distance from center of an anchor shaft to the edge of concrete:

$$\Psi_{ed,N} = 1 \ \text{if}\ c_{a,min} \geq 1.5h_{ef} \quad \text{Equation 7-25}$$

$$\Psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \ \text{if}\ c_{a,min} < 1.5h_{ef} \quad \text{Equation 7-26}$$

$\Psi_{c,N}$=1.25 can be used for regions where analysis indicates no cracking at service load levels; otherwise, refer to Appendix D ACI 318-18.

$\Psi_{cp,N}$ =1.0 is used for cast in place anchors.

iii) $\phi_t N_{pn}$ is the available pullout of anchor in tension.

$$\phi_t N_{pn} = 0.7\Psi_{c,p} N_p \qquad \text{Equation 7-27}$$

$\Psi_{c,p}$=1.4 is used based for no cracking at service load levels.

$$N_p = 8A_{brg} f_c' \qquad \text{Equation 7-28}$$

iv) The available concrete side-face blowout strength of a headed anchor in tension $\phi_t N_{sb}$ or $\phi_t N_{sbg}$ for a single anchor and a group of anchors is evaluated for anchors with deep embedment close to an edge when c_{a1}<0.4h_{ef}

For a single anchor:

$$\phi_t N_{sb} = 0.7 \times 160 c_{a1} \sqrt{A_{brg}} \sqrt{f_c'} \qquad \text{Equation 7-29}$$

If c_{a2}<3c_{a1} for the single headed anchor, the value of $\phi_t N_{sb}$ is multiplied by the factor (1 + c_{a2}/c_{a1})/4 where 1.0 ≤ c_{a2}/c_{a1} ≤ 3.0

For a group of anchors with anchor spacing <6c_{a1}

$$\phi_t N_{sbg} = 0.7 \times 160 c_{a1} \sqrt{A_{brg}} \sqrt{f_c'} \left(1 + \frac{S}{6c_{a1}}\right) \qquad \text{Equation 7-30}$$

Figure 7-1 Parameters of steel-headed anchors to calculate axial strength

{{

}}$^{2(a),(c)}$

7.7.4 Studs Subjected to Shear and Tension

Studs subject to combined tensile strength and shear is calculated as follows:

1. If concrete breakout strength in shear is not a governing limit and where c_{a1}≥1.5h_{an} and S_{an}≥3h_{an}, the interaction Equation 7-31 is satisfied. Where Q_{rt}

and Q_{rv} are the required tensile strength per anchor, Q_{cv} and Q_{ct} are as determined in Section 7.7.2 and Section 7.7.3, respectively.

$$\left[\left(\frac{Q_{rt}}{Q_{ct}}\right)^{5/3} + \left(\frac{Q_{rv}}{Q_{cv}}\right)^{5/3}\right] \leq 1.0 \qquad \text{Equation 7-31}$$

2. If concrete breakout strength in shear is a governing limit state or where c_{fe}<1.5h_{an} or if S_{an}<3h_{an}

   a. Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318-08, Equation 7-30 is used, where Q_{cv} and Q_{ct} are from Equation 7-4 and Equation 7-18, respectively.

   b. Otherwise, the interaction is to be calculated per Appendix D of ACI 318-08. Also, the interaction of shear and tension are only checked for the case when Q_{rt} >0.2Q_{ct} and Q_{rv}>0.2Q_{rv}, then the interaction is checked per Equation 7-32. Q_{cv} and Q_{ct} are as determined in Section 7.7.2 and Section 7.7.3, respectively.

$$\frac{Q_{rt}}{Q_{ct}} + \frac{Q_{rv}}{Q_{cv}} \leq 1.2 \qquad \text{Equation 7-32}$$

7.8 Welds

The available strength for welds is determined in accordance with Chapter J of ANSI/AISC 360-16 as explained in this section.

7.8.1 Effective Area

The effective area of groove and fillet welds is the length of the weld times the effective throat.

For fillet welds in holes and slots, the effective length is the length of the centerline of the weld along the center of the plane through the throat.

The effective shearing area of plug and slot welds is considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

7.8.2 Effective Throat

The effective throat of welds $t_{th\text{-}eff}$ is determined in Chapter J of AISC 360-16 as discussed below:

1. The effective throat of a full penetration (CJP) weld is the thickness of the thinner parts joined.
2. For partial penetration groove weld, the effective throat is dependent on the process used and the weld position. The design drawings either indicate the

effective throat required or the weld strength required. For more information, refer to AISC 360-16.

3. For flare groove weld (bevel groove and V-groove), consistent with AISC 360-16 and as shown in Table 7-1, the effective weld throat is as follows:

   a. When the weld is filled flush to the surface of a round bar or a 90-degree bend in a formed section, the effective throat is as shown in Table 7-1, unless other effective throats are demonstrated by tests. R used in Table 7-1 is the radius of joint surface.

   b. The effective throat of flare groove welds filled less than flush is as shown in Table 7-1, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

4. For fillet welds, the effective throat of a fillet weld is the shortest distance from the root to the face of the diagrammatic weld.

Table 7-1 Effective weld throats of flare groove welds

Welding Process	Flare Bevel Groove	Flare V-Groove
GMAW and FCAW-G	{{	
SMAW and FCAW-S		
SAW		}}2(a),(c)

Note:
GMAW: gas metal arc welding
FCAW-G: gas-shielded flux cored arc welding
SMAW: shielded metal arc welding
FCAW-S: self-shielded flux cored arc welding
SAW: submerged arc welding

7.8.3 Available Strength

The available strength of welded joints is the lower value of the base material strength determined according to the limit states of tensile rupture and shear rupture and the weld metal strength determined according to the limit state of rupture as follows:

$$\phi R_n = \min(F_{nbm}A_{bm}, F_{nw}A_{we}) \quad \text{Equation 7-33}$$

The available strength of welded joint is per AISC 360-16 as shown in Table 7-2, where U is the shear lag factor for the base metal according to AISC 360-16.

Table 7-2 Available strength of welded joints (ksi)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ	Nominal Stress (F_{nbm} or F_{nw})	Effective Area (A_{bm} or A_{we})
CJP GROOVE WELDS				
Tension or compression normal to weld axis Shear	Strength of the joint is controlled by the base metal			
Tension or compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld are not considered in design of welds joining the parts.			
PARTIAL- JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE V-GROOVE AND FLARE BEVEL GROOVE WELDS				
Tension normal to weld axis	Base	{{	F_y	A_nU
	Weld		$0.6F_{EXX}$	$L_w \times t_{th\text{-}eff}$
Compression connections of members designed to bear other than columns as described in Section J1.4(2) of AISC 360-16	Base		F_y	A_g
	Weld		$0.6F_{EXX}$	$L_w \times t_{th\text{-}eff}$
Compression connections not finished-to-bear	Base		F_y	A_g
	Weld	}}[2(a),(c)]	$0.9F_{EXX}$	$L_w \times t_{th\text{-}eff}$
Tension or compression parallel to weld axis	Tension or compression in parts joined parallel to a weld does not need to be considered in the design			
Shear	Base		min($0.6F_yA_{gv}$, $0.75\times0.6F_uA_{nv}$)	
	Weld	{{ }}[2(a),(c)]	$0.6F_{EXX}$	$L_w \times t_{th\text{-}eff}$
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS				
Shear	Base		min($0.6F_yA_{gv}$, $0.75\times0.6F_uA_{nv}$)	
	Weld	{{ }}[2(a),(c)]	$0.6F_{EXX}$	$L_{w\text{-}eff} \times t_{th\text{-}eff}$
Tension or compression parallel to weld axis	Tension or compression in parts joined parallel to a weld does not need to be considered in the design			
PLUG AND SLOT WELDS				
Shear parallel to faying surface on the effective area	Base		min($0.6F_yA_{gv}$, $0.75\times0.6F_uA_{nv}$)	
	Weld	{{ }}[2(a),(c)]	$0.6F_{EXX}$	nominal cross-sectional area of the hole or slot in the plane of the faying surface

7.9 Bolts and Threaded Parts

This section covers the design of snug tight or pretension bolts. For information about the design of slip-critical connections, refer to AISC 360-16.

7.9.1 Tensile and Shear Strength

The available tensile or shear strength of a snug tightened or pretension high strength bolts and threaded parts are given in AISC 360-16 as shown below:

$$\phi R_n = 0.75 F_n A_b \qquad \text{Equation 7-34}$$

F_n is the nominal tensile stress F_{nt} (ksi) or shear stress F_{nv} (ksi) from AISC 360-16.

7.9.2 Combined Tensile and Shear Strength

The available tensile strength of a bolt subjected to combined tension and shear is determined according to AISC 360-16 as shown below:

$$\phi R_n = 0.75 F'_{nt} A_b \qquad \text{Equation 7-35}$$

where F'_{nt} is the nominal tensile stress modified to include the effect of shear stress as shown in Equation 7-36.

$$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt} \qquad \text{Equation 7-36}$$

where f_{rv} is the required shear stress (ksi)

7.9.3 Bearing Strength at Bolt Holes

The available bearing strength at bolt holes is determined per AISC 360-16 as shown below:

1. For a bolt in a connection with standard, oversized, and short-slotted holes, independent of the direction of loading, or with a long-slotted hole with the slot parallel to the direction of the bearing force:
    a. When deformation at the bolt hole at service load is a design consideration

    $$\phi R_n = 0.75(1.2 l_c t F_u \le 2.4 d_b t F_u) \qquad \text{Equation 7-37}$$

    b. When deformation at the bolt hole at service load is not a design consideration

    $$\phi R_n = 0.75(1.5 l_c t F_u \le 3.0 d_b t F_u) \qquad \text{Equation 7-38}$$

2. For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force

$$\phi R_n = 1.0(1.0 l_c t F_u \le 2.0 d_b t F_u) \qquad \text{Equation 7-39}$$

where l_c is 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,

t is the thickness of connected material,

d_b is the nominal bolt diameter, and

F_u is the specified minimum tensile strength of the connected material

7.10 Compression Transfer via Direct Bearing on Concrete

Where force is transferred in an encased or filled composite member by direct bearing from internal bearing mechanisms, the available bearing strength of the concrete for the limit state of concrete crushing is determined in accordance with AISC 360-16 and shown below:

$$\phi R_n = 0.65(1.7 f_c' A_l) \qquad \text{Equation 7-40}$$

where A_l is the loaded area of concrete

7.11 Shear Friction Load Transfer Mechanism

The available strength is determined in accordance with ACI 349-13 as explained in this section. The shear reinforcement to transfer shear friction cannot be accounted for to transfer net tension

1. where shear-friction reinforcement is perpendicular to the shear plane:

$$\phi V_n = 0.75 A_{vf} f_y \mu \leq 0.75 \min\left(0.2 f_c' A_c,\ (480 + 0.08 f_c') A_c,\ 1600 A_c\right) \qquad \text{Equation 7-41}$$

2. where shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement

$$\phi V_n = 0.75 A_{vf} f_y (\mu \sin\alpha + \cos\alpha) \leq 0.75 \min\left(0.2 f_c' A_c,\ (480 + 0.08 f_c') A_c,\ 1600 A_c\right) \qquad \text{Equation 7-42}$$

Where,

α is the angle between shear-friction reinforcement and shear plane,

A_c is area of concrete section resisting shear transfer,

F_y is the specified minimum yield stress of the shear friction reinforcement ≤ 60ksi, and

μ is the coefficient of friction taken from ACI 349-13 as follows:

a. for concrete placed monolithically, $\mu=1.4$
b. for concrete placed against hardened concrete with surface intentionally roughened to a full amplitude of approximately 1/4 in. as specified in ACI 349-13, $\mu=1.0$

c. for concrete placed against hardened concrete not intentionally roughened, $\mu=0.5$

d. concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars ACI 349-13, $\mu=0.7$.

7.12 Embedded Shear Lugs and Shapes

The available strength and reduction factor are determined in accordance with Appendix D of ACI 349-13.

7.12.1 Bearing Strength

The available bearing strength for concrete against shear lugs is calculated as described in ACI 349-13 and shown in the equation below.

$$\phi f_b = 1.3\phi f_c' A_{lug} \qquad \text{Equation 7-43}$$

With ϕ = 0.65 for anchor controlled by concrete bearing and A_{lug} is the beading area of the shear lug.

7.12.2 Shear Strength

The available shear strength of embedded shear lugs bearing toward a free edge is per Appendix D of ACI 349-13, as:

$$\phi V_n = 4\phi\sqrt{f_c'} A_{sh} \qquad \text{Equation 7-44}$$

With ϕ = 0.80 for embedded shear lugs toward free edge and A_{sh} is the bearing area defined by projecting a 45-degree plane from bearing edges of the shear lug to the free surface. The bearing area of the shear lug is excluded from the projected area.

7.13 Anchor Rods

This section includes rules for cast-in anchor rods only. Rules for headed anchor studs are discussed in Section 7.7.1.

Refer to Appendix D of ACI 349-13 for information regarding post-installed anchors. The available strength is determined from Appendix D.

7.13.1 Available Strength in Tension

The available strength in tension is the minimum of ($\phi_t N_{sa}$, $\phi_t N_{cb}$ or $\phi_t N_{cbg}$, $\phi_t N_{pn}$, $\phi_t N_{sb}$ or $\phi_t N_{sbg}$)

1. The available strength in tension for anchor rods $\phi_t N_t$ is per Appendix D of ACI 349-13:

$$\phi_t N_{sa} = 0.75 n_a A_{se} f_{uta} \quad \text{Equation 7-45}$$

   F_{uta} is the min of (1.9f_{ya}, 125ksi)

   ϕ is the strength reduction factor taken from Appendix D of ACI 349-13 for anchor governed by strength of a ductile steel element under tension loads.

2. The available concrete breakout strength of anchor in tension $\phi_t N_{cb}$ or $\phi_t \mathrm{N}_{\mathrm{cbg}}$ per Appendix D of ACI 349-13 and in Equation 7-46 and Equation 7-47 for single anchor and a group of anchors respectively:

$$\phi_t N_{cb} = 0.70 \frac{A_{Nc}}{A_{Nco}} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \quad \text{Equation 7-46}$$

$$\phi_t N_{cbg} = 0.70 \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \quad \text{Equation 7-47}$$

   ϕ_t is the strength reduction factor taken from Appendix D of ACI 349-13 for cast in headed studs governed by concrete breakout, Condition B.

   where,

   - A_{Nco} is the projected concrete failure area of a single anchor with an edge distance equal to or greater than 1.5h_{ef}:

$$A_{Nco} = 9 h_{ef}^2 \quad \text{Equation 7-48}$$

   - h_{ef} is the effective embedment depth of the anchor. Where anchors are located less than 1.5h_{ef} from three or more edges, the value of h_{ef} used in this section is the greater of $c_{a,max}$ /1.5 and one-third of the maximum spacing between anchors within the group.
   - A_{Nc} is the projected concrete failure area of a single anchor or group of anchors that is approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward 1.5h_{ef} from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors.

- N_b is the basic concrete breakout strength of a single anchor in tension in cracked concrete for cast-in anchors:

$$N_b = 24\sqrt{f_c'}h_{ef}^{1.5} \qquad \text{Equation 7-49}$$

- The modification factor for an anchor group loaded eccentrically in tension $\Psi_{ec,N}$ is calculated as:

$$\Psi_{ec,N} = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)} \leq 1.0 \qquad \text{Equation 7-50}$$

  where e'_N is the eccentricity calculated as follows:

  - If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension are considered when determining the eccentricity e'_N for use in Equation 7-50.
  - In the case where eccentric loading exists about two axes, the modification factor $\Psi_{ec,N}$ is computed for each axis individually and the product of these factors used as $\Psi_{ec,N}$ in Equation 7-50.

- The modification factor for edge effects for single anchor or anchor groups loaded in tensions is given below, where $c_{a,min}$ is the minimum distance from center of an anchor shaft to the edge of concrete:

$$\Psi_{ed,N} = 1 \text{ if } c_{a,min} \geq 1.5h_{ef} \qquad \text{Equation 7-51}$$

$$\Psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \text{ if } c_{a,min} < 1.5h_{ef} \qquad \text{Equation 7-52}$$

- For $\Psi_{c,N}$=1.25 assuming no cracking under service load and $\Psi_{cp,N}$=1.0 for cast in-place anchors.

3. The available pullout strength of anchor in tension $\phi_t N_{pn}$ of a single anchor

$$\phi_t N_{pn} = 0.7\Psi_{c,p}N_p \qquad \text{Equation 7-53}$$

   $\Psi_{c,p}$=1.4 is used based on no cracking at service load levels. Refer to Appendix D of ACI 349-13 (Reference 10.1.4) otherwise.

$$N_p = 8A_{brg}f_c' \qquad \text{Equation 7-54}$$

4. The available concrete side-face blowout strength of a headed anchor in tension $\phi_t N_{sb}$ or $\phi_t N_{sbg}$ for a single anchor and a group of anchors need to be evaluated for anchors with deep embedment close to an edge when c_{a1}<0.4h_{ef}

For a single anchor:

$$\phi_t N_{sb} = 0.7 \times 160 c_{a1} \sqrt{A_{brg}} \sqrt{f_c'} \qquad \text{Equation 7-55}$$

If $c_{a2} < 3c_{a1}$ for the single headed anchor, the value of $\phi_t N_{sb}$ is multiplied by the factor $(1 + c_{a2}/c_{a1})/4$ where $1.0 \le c_{a2}/c_{a1} \le 3.0$

For a group of anchors with anchor spacing $<6c_{a1}$

$$\phi_t N_{sbg} = 0.7 \times 160 c_{a1} \sqrt{A_{brg}} \sqrt{f_c'} \left(1 + \frac{S}{6c_{a1}}\right) \qquad \text{Equation 7-56}$$

7.13.2 Available Strength in Shear

The shear strength of the anchor rod is calculated using Appendix D of ACI 349-13.

The shear strength of the steel-headed anchors Q_{cv} are calculated as the minimum of ($\phi_v V_{sa}$, $\phi_v V_{cb}$ or $\phi_v V_{cbg}$, $\phi_v V_{cp}$ or $\phi_v V_{cpg}$):

1. $\phi_v V_{sa}$, steel strength of anchor in shear using Appendix D of ACI 349-13.

$$\phi_v V_{sa} = 0.65 n_a A_{se} f_{uta} \qquad \text{Equation 7-57}$$

where

n is the number of anchors in the group,

A_{se} is the effective cross-sectional area of a single anchor in shear,

f_{uta} is taken as the max($1.9f_{ya}$,125ksi), and

ϕ_v is taken as 0.65 per ACI 349-13.

Note: Per Appendix D of ACI 349-13, where anchors are used with built-up grout pads, the shear strength from Equation 7-57 is multiplied by a 0.80 factor.

2. $\phi_v V_{cb}$ or $\phi_v V_{cbg}$, concrete breakout strength of anchor in shear using Appendix D of ACI 349-13, where the available concrete breakout strength, $\phi_v V_{cb}$ or $\phi_v V_{cbg}$

   a. For shear force perpendicular to the edge on a single anchor

$$\phi V_{cb} = 0.70 \frac{A_{Vc}}{A_{Vco}} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b \qquad \text{Equation 7-58}$$

   b. For shear force perpendicular to the edge on a group of anchors

$$\phi V_{cbg} = 0.70\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b \qquad \text{Equation 7-59}$$

c. For shear force parallel to an edge, $\phi_v V_{cb}$ or $\phi_v V_{cbg}$ is permitted to be twice the value of the shear force determined from Equation 7-58 or Equation 7-59, respectively, with the shear force assumed to act perpendicular to the edge and with $\Psi_{ed,V}$ taken equal to 1.0.

d. For anchors located at a corner, the limiting nominal concrete breakout strength is determined for each edge and the minimum value is used.

A_{Vco} is the projected area for a single anchor in a deep member with a distance from edges equal or greater than $1.5c_{a1}$ the direction perpendicular to the shear force. It is permitted to evaluate A_{Vco} as the base of a half-pyramid with a side length parallel to the edge of $3c_{a1}$ and a depth of $1.5c_{a1}$

$$A_{Vco} = 4.5(c_{a1})^2 \qquad \text{Equation 7-60}$$

c_{a1} is the distance from the center of an anchor shaft to the edge of concrete in one direction

A_{Vc} is the projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. A_{Vc} does not exceed nA_{Vco}, where n is the number of anchors in the group.

V_b is the basic concrete breakout strength in shear of a single anchor in cracked concrete:

$$V_b = 7\left(\frac{h_{ef}}{d_s}\right)^{0.2}\sqrt{d_s}\sqrt{f_c'}(c_{a1})^{1.5} \qquad \text{Equation 7-61}$$

Where d_s is the outside diameter of the anchor rod. h_{ef} is the effective embedment length (refer to Appendix D of ACI 349-13).

The modification factor for anchor groups loaded eccentrically in shear is given in Equation 7-62, where e'_V is the distance between resultant shear load on a group of anchors loaded in shear in the same direction and the centroid of the group of anchors loaded in shear in the same direction:

$$\Psi_{ec,V} = \frac{1}{\left(1 + \frac{2e'_V}{3c_{a1}}\right)} \le 1 \qquad \text{Equation 7-62}$$

If the loading on an anchor group is such that only some anchors are loaded in shear in the same direction of the free edge, only those anchors that are loaded in shear in the direction of the free edge are considered when determining the eccentricity of e'_{V}.

The modification factor for edge effect for a single anchor or group of anchors loaded in shear is given in Equation 7-12 or Equation 7-13 based on c_{a2}, where c_{a2} is the distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}.

$$\Psi_{ed,V} = 1 \ \mathit{if} \ c_{a2} \geq 1.5c_{a1} \qquad \text{Equation 7-63}$$

$$\Psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \quad \mathit{if} \ c_{a2} < 1.5c_{a1} \qquad \text{Equation 7-64}$$

For modification factor $\Psi_{c,V}$, it is assumed the anchor rods are located in a region of a concrete member where analysis indicates no cracking under the service load levels. That is $\Psi_{c,V}$ = 1.4. Refer to Appendix D of ACI 349-13 otherwise.

The modification factor for anchors located in a concrete member where $h_a < 1.5c_{a1}$, $\Psi_{h,V}$ is computed as

$$\Psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0 \qquad \text{Equation 7-65}$$

3. $\phi_v V_{cp}$ or $\phi_v V_{cpg}$, concrete pryout strength of anchor in shear using Appendix D of ACI 349-13

   a. For a single anchor

$$\phi V_{cp} = 0.70k_{cp}N_{cb} \qquad \text{Equation 7-66}$$

   b. For a group of anchors

$$\phi V_{cp} = 0.70k_{cp}N_{cbg} \qquad \text{Equation 7-67}$$

where

k_{cp} = 1.0 for h_{ef} < 2.5 in.

k_{cp} = 2.0 for $h_{ef} \geq$ 2.5 in.

N_{cb} and N_{cbg} are per Equation 7-46 and Equation 7-47.

7.13.3 Interaction of Tensile and Shear Forces

Based on the factored shear and tensile force applied to a single anchor or a group of anchors V_{ua} and N_{ua}, respectively, the interaction for tensile and shear forces are checked as follows:

1. If $V_{ua} \leq 0.2\ \phi V_n$ full strength in tension is permitted, that is: $\phi N_n \geq N_{ua}$
2. If $N_{ua} \leq 0.2\ \phi N_n$ full strength in shear is permitted, that is: $\phi V_n \geq V_{ua}$
3. Otherwise,

$$\frac{N_{ua}}{\phi B_n} + \frac{V_{ua}}{\phi V_n} \leq 1.2 \qquad \text{Equation 7-68}$$

8.0 Design Methodology for Seismic Category I and Category II Reinforced Concrete Structures

8.1 General Information

Finite element models are generally used for the seismic analysis of nuclear power plants. The design analysis is usually performed following one of two methods. One method is the element-based approach, in which the demand consists of finite element stress resultants (i.e., element forces and moments per unit width). The second method is the member-based or section cut-based approach, in which the demand consists of forces and moments obtained at member cross sections or section cuts (Reference 10.1.27).

The contour plots of stress resultants from finite element analyses are used to identify critical areas for design. Once the stress resultants are obtained, the design may be performed by using either one of the aforementioned approaches.

The element-based design approach is mesh-dependent and can be overly conservative at certain locations (e.g., at small finite elements, corners, near concentrated forces, and openings).

Forces obtained at a section cut are resultant forces acting along the length of the member section. Thus, the section cut forces take into account the redistribution of peak finite element stresses along the length of the member section, which normally occurs in RC structures. The section cut approach yields design demands that are more realistic.

In this design methodology, the required strengths are calculated at section cuts along critical member sections. The demand calculated this way is consistent with ACI 349 in which members “shall be designed to have design strengths at all sections at least equal to the required strengths…,” and is also consistent with the code based equations for design capacities.

Time histories of seismic and static member forces are calculated using ANSYS finite element models and the process described in Section 4.0. To simplify the process of calculating and combining seismic and static forces and moments acting on cuts, the models used for seismic and static analyses have the same finite element mesh for RC members.

For both static and seismic analyses, section design forces and moments are determined for cuts located along a row of nodes as described in Section 8.4.2. Cuts may be in either horizontal or vertical direction.

Nodal forces, acting on the cut nodes, coming from the elements on one side of the cut are calculated during the ANSYS solution phase. Next, using the ANSYS postprocessor, the section cut forces and moments are calculated by summing the nodal forces and moments (from the elements on one side of the cut) about a point at the center of gravity (CG) of the cut. The static forces and moments at the CG and seismic time histories of forces and moments at the CG are written to text files. The load combinations use static forces with seismic forces on a time step basis for the entire time-history duration. Load combinations and design checks are then performed outside ANSYS.

For seismic analysis, dynamic analysis is performed in the frequency domain (i.e. harmonic response analysis) using the soil library method described in TR-0118-58005-P. The transfer function for calculating nodal forces acting on the cut are obtained during the ANSYS harmonic solution phase. The transfer functions for the section cut forces and moments are calculated using the ANSYS postprocessor. The harmonic analysis is performed for a subset of frequencies of the entire set in the frequency domain. From this subset of results, transfer functions for section cut forces are interpolated for all frequencies in the frequency domain, convolved with input motions (control motions), and transformed to the time domain using the FFT. The process of interpolating transfer function, convolving with input time motions, and transforming to the time domain is performed within or outside ANSYS. The final result is the time history of section cut forces and moments written to text files for subsequent use in load combinations and design checks.

8.2 Purpose

The objective of this section is to develop a methodology for the design of Seismic Category I and II concrete structures, according to the requirements of ACI 349. Pertinent requirements are also included from Regulatory Guide 1.142 (Reference 10.1.6).

This methodology applies to the design of the RC members of a representative SMR Reactor Building, CRB, and RWB. The design methodology describes the building load path and design actions on the main structural members. It also describes the different required strengths for RC members, including guidelines to determine critical sections in slabs, basemats, beams and columns, where section cuts are to be provided.

The RC members addressed are floor slabs and beams, gravity columns, and the foundation basemats.

The SC walls, composite roof or metal decks, and other supporting steel structures are not covered.

The connections between RC slabs and SC walls are addressed in Section 7.0 of the design methodology for SC wall connections.

8.3 Lateral and Gravity Load-Resisting Systems

Seismic Category I and II buildings are often constructed as RC bearing wall-type buildings. In the representative buildings described herein, the main structural components are thick SC walls located mainly around the perimeter of the buildings and RC slabs at different levels. Slabs connecting walls at different floor levels can include beams cast together with the slabs (T-beams).

In accordance with ACI 349, the main seismic load-resisting system in both N-S and E-W directions is comprised of shear walls (SC walls) and concrete diaphragms consisting of the floor slabs and the roof, as shown in Figure 8-1. During the SSE, seismic forces are transferred to the walls by in-plane bending and shear of the diaphragms and back to the foundation by in-plane bending and shear of the walls. A small portion of the seismic force is also transferred through out-of-plane bending of the walls and out-of-plane bending of the slabs, referred to as "slab-wall frame action," but this is not significant.

The gravity load-resisting system is comprised of floor slabs, T-beams, SC walls, isolated RC columns, and foundation basemats. Due to gravity loads, floor slabs and basemats are subject to out-of-plane moment and out-of-plane shear. These forces will increase or decrease due to vertical seismic loads.

The embedded portions of exterior walls and basemats are also subject to static and dynamic soil pressures, and hydrostatic water pressures, including buoyancy. Interior walls around the RXB pool area are subject to hydrostatic and hydrodynamic water pressures. These actions increase the out-of-plane moment and the out-of-plane shear on basemats.

8.3.1 Floor Slabs and Basemats

Floor slabs are subject to out-of-plane forces, primarily due to gravity loads and frame action, and in-plane diaphragm forces, due to seismic loads.

Slabs subjected to in-plane diaphragm forces due to seismic effects are designed using the special seismic requirements for diaphragms of ACI 349 Section 21.11 (Reference 10.1.4). Since frame action is limited by the presence of the shear walls, slabs subjected to out-of-plane forces are designed using the requirements for conventional RC members of ACI 349 Chapters 1 through 19.

Basemats are subjected to out-of-plane forces and in-plane forces. The out-of-plane forces are due to static and dynamic soil bearing pressure, and pressure due to buoyancy. Also, rocking of the walls due to seismic loads generates additional out-of-plane moment and out-of-plane shear along the basemat thickness The in-plane forces in the basemat are due to seismic base shears from the structural walls.

Basemats are designed using the requirements for conventional RC members of ACI 349 Chapters 1 through 19, and the special requirements for foundations resisting seismic loads from ACI 349 Section 21.12.

8.3.2 T-Beams

The main function of the T-beams is to resist loads caused by out-of-plane flexure of the slabs to which they attach. They are designed as per ACI 349 for conventional flexural members with flexure about their strong axis.

8.3.3 Columns

The primary function of columns is to support the gravity load of the floor above them. Thus, they are designed using ACI 349 provisions for conventional axial-flexural members (ACI 349 Chapters 1 through 19).

Figure 8-1 In-plane (diaphragm) and out-of-plane actions in floor slabs (from Reference 10.1.21)

{{

}}2(a),(c)

8.4 Design Methodology

8.4.1 Introduction

Specific details on how to obtain the required strengths from the FEA models are given in the following sections.

ACI 349-13 refers to ACI 318-08 (Reference 10.1.16) in several sections. In this topical report, when ACI 318 is called out by ACI 349, the ACI 349 section is cited in most situations, unless specific information is used from ACI 318. From this point forward in this topical report, "ACI 318" will refer to ACI 318-08.

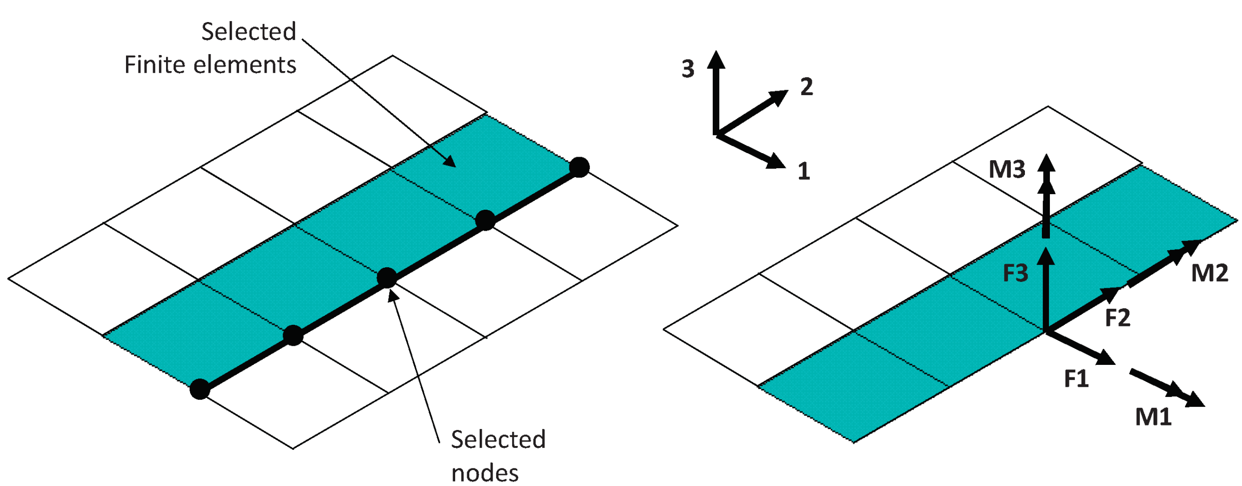
8.4.2 Section Cut Forces

Section cuts are created in FEA models by selecting the elements at one side of the desired cut and the nodes along the cut (Figure 8-2). Section cut forces are then calculated by the FEA software as the summation of the nodal forces along the cut. The force summation is performed at the centroid of the section cut which is consistent with the reference point used in the calculation of the internal moments in the P-M interaction diagrams.

Section cut forces are usually provided in the global model coordinate system. However, for design, it is more convenient to calculate section cut forces with respect to a local coordinate system such that the calculated force components correspond to the same action for any arbitrarily oriented section cut. For example, the section cut forces for the slab panel shown in Figure 8-2 are reported in the local section cut coordinate system with the following output demand components:

- F1, the axial load (P)
- F2, the in-plane shear (V)
- F3, the out-of-plane shear (V_z)
- M1, the torsional moment (T)
- M2, the out-of-plane moment (M)
- M3, the in-plane moment (M_z)

Figure 8-2 Section cut at a slab panel modeled with shell elements and force resultants in the section cut local coordinate system



For line elements (e.g., columns), section cuts are created by selecting the element along the cut and the node at the cut (e.g., Figure 8-3).

For T-beams, ACI requires the inclusion of an effective slab width defined by the requirements in ACI 318 Section 8.12. This effective slab width is only required when the slab portion above the beam is in compression. Conservatively for beam design, the effective widths are used in situations when evaluating the beam demand. In this case, the section cut also includes the finite elements corresponding to the effective slab width and the slab nodes along the width, as shown in Figure 8-4.

The demand types, based on the section cut local coordinate system shown in Figure 8-3 and Figure 8-4 are shown below. It is assumed that axis 3 is the strong-axis bending of columns and beams in floor slabs.

- F1, the axial load (P)
- M3 and V2 are the strong-axis bending and shear in columns and out-of-plane moment and shear in T-beams,
- M2 and V3 are the weak-axis bending and shear in columns and in-plane moment and shear in T-beams,
- M1 is the torsional moment

Figure 8-3 Section cut for columns and section cut local coordinate system

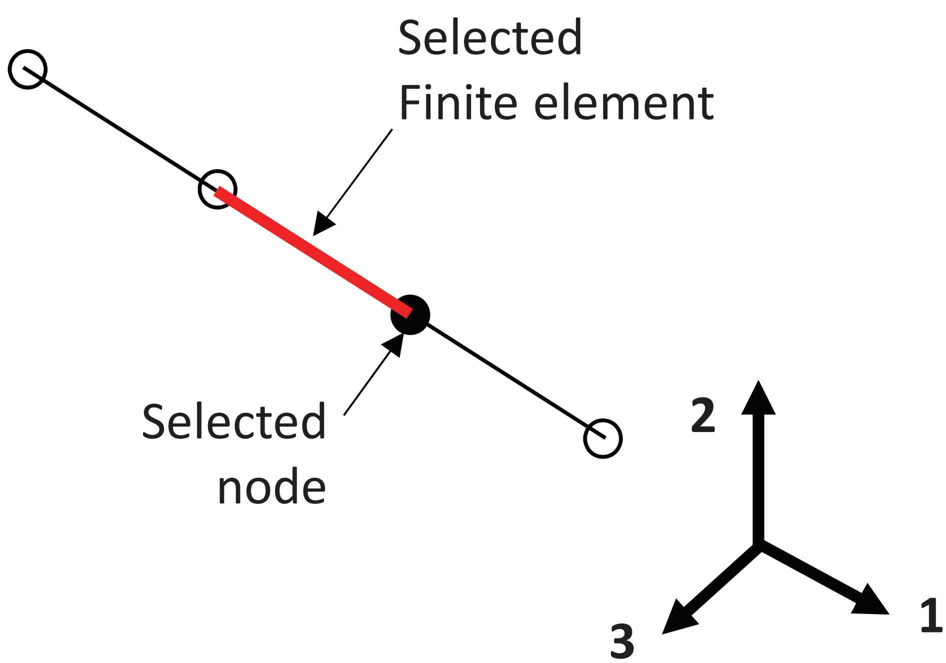
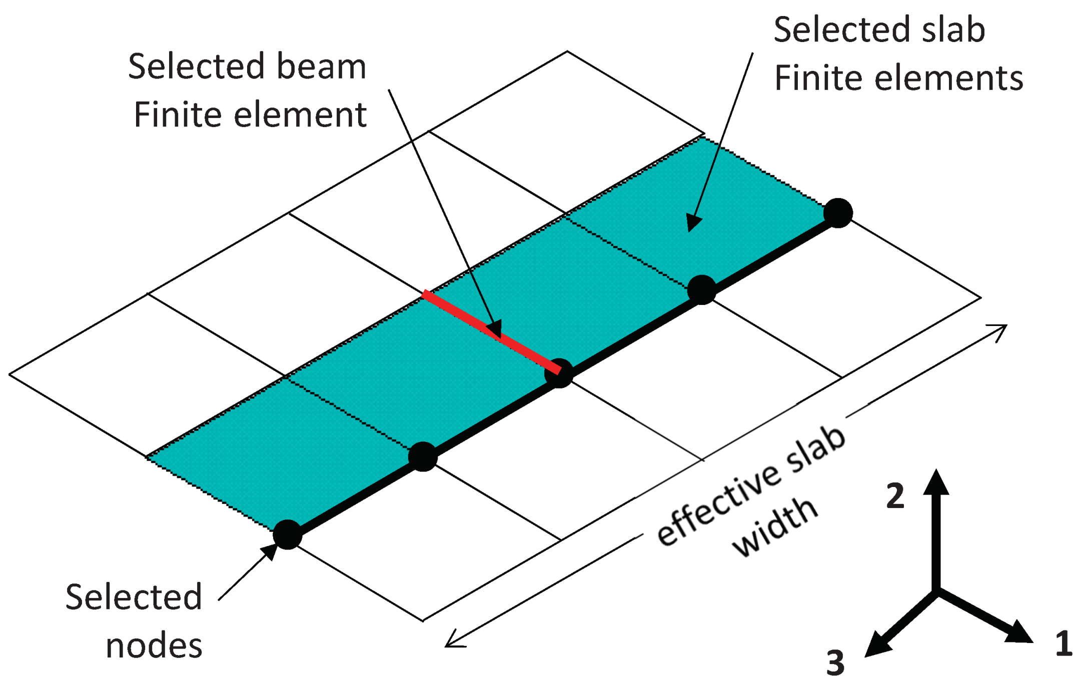


Figure 8-4 Section cut for T-beams and section cut local coordinate system



8.5 Required Strengths for Slab and Basemat Design

From the six demand types obtained at a section cut, the required strengths for slab and basemat design are:

- Axial load, P, and out-of-plane moment, M, are used to design for axial force-moment (P-M) interaction.
- Axial load, P, and out-of-plane shear, V_z, are used to design for axial force-shear (P-V_z) interaction
- The in-plane shear, V, is used to design for shear and shear-friction at slab-wall interfaces

The in-plane moment, M_z, can be represented as a force couple or chord forces located near the diaphragm edges. The effect of chord forces is an increase in the axial load, P, at these locations. Thus, by calculating the axial load at discrete section cuts at critical member regions, including regions near the diaphragm edges, the effect of the in-plane moment is included in the diaphragm design.

Similarly, the effect of diaphragm collector forces is an increase in the axial load, P, at the end of discontinuous walls. Thus, by calculating the axial load at discrete section cuts at critical member regions, including regions at the end of discontinuous walls, the effect of the collector forces is included in the diaphragm design.

As discussed in Reference 10.1.18, the effect of the torsional moment, T, is an increase in the out-of-plane shear force, V_z. In the design process, this effect is considered by calculating out-of-plane shear at discrete section cuts at critical regions which also include the areas with high torsion.

Sections 8.5.1, 8.5.2, and 8.5.3 provide general guidelines to determine critical locations where largest demand is expected in slabs subjected to vertical and lateral loading. These critical locations are based on elastic distribution of moments in a two-way slab supported at its four edges and subject to uniform vertical and lateral loading. The presence of nearby openings and non-uniform loading, however, may affect the location of critical sections and may require evaluation of additional sections in a given slab.

For clarity, critical locations are shown separately for each demand type resulting from out-of-plane and in-plane actions. Critical locations for in-plane actions depend on the direction of the horizontal seismic force and, hence, two sets of section cut locations are shown for each input motion direction. Critical locations are also shown around openings.

The critical section cut locations defined in Sections 8.5.1, 8.5.2, and 8.5.3 are considered as approximate and the minimum set of locations for demand extraction. Therefore, additional section cuts are used to investigate the possibility of larger demands outside these critical locations. The length of section cuts, defined in Section 8.5.1, for out-of-plane moment and out-of-plane shear is conservatively limited to three times the member thickness to capture the effect of concentrated loads. In the absence of concentrated loads (i.e., slab subject to uniform loading) larger sections can be justified. On the other hand, if the critical section is limited by openings that result in a section cut length less than three times the thickness, then that smaller section should be used. Thus, three times the thickness is not considered as a fixed value, instead it is used to average design loads to avoid unrealistic excessive conservatism. In all cases, the design engineer needs to justify the use of three times or other averaging lengths.

The additional section cut locations are identified using finite element analysis results in the form of stress resultant contour plots, as described in Section 8.5.4. In this approach, the actual stress distributions, due to out-of-plane moment and out-of-plane shear demands, are used to identify critical regions and to determine section cut lengths. If needed, a justification can be provided per Section 8.5.4 to make section cut lengths longer than three times the member thickness. In general, the section cut lengths determined through finite element stress resultants need not be less than three times the member thickness unless the stress resultant changes sign along the cut or it is limited by openings. In stress resultant contour plots, stress concentrations are expected to be present in a few finite elements located adjacent to openings or located where finite element mesh or component geometry includes abrupt changes. These stress concentrations are not representative as concrete will crack and distribute the localized stresses over a larger area.

8.5.1 Section Cuts for Out-of-Plane Forces

The out-of-plane forces in floor slabs correspond to out-of-plane moment, M, and out-of-plane shear, V_z, generated by vertical loads (e.g., gravity, buoyancy, vertical seismic loads, etc.) and lateral loads (e.g., seismic load) due to frame action.

Considering the four-edged fixed slab shown in Figure 8-5, the out-of-plane moments due to gravity load (assuming uniform load) for the middle strip along the north-south direction are shown in Figure 8-5c. As shown in the figure, the largest negative moment occurs at the face of the supports and the largest positive moment occurs at the center of the span. With respect to the east-west direction, the magnitude of these moments is the largest at the middle and diminishes towards the corners. Similar behavior is observed for the moments along the east-west direction although they became smaller as the east-west span increases with respect to the north-south span (i.e., as the slab approaches to a one-way slab).

For the same slab strip shown in Figure 8-5a, the out-of-plane moments due to frame action along the north-south direction are shown in Figure 8-5d. As shown in the figure, the largest moments occur at the face of the supports and their sign changes depending on the sway direction. Because these moments are generated by the out-of-plane movement of walls, their magnitude is also the largest around the middle of the east-west span.

Figure 8-5b also shows the distribution of gravity loads to supporting walls based on the yield-line theory. Based on this distribution, it can be concluded that the out-of-plane shear forces are concentrated around the center of each of the four slab supports.

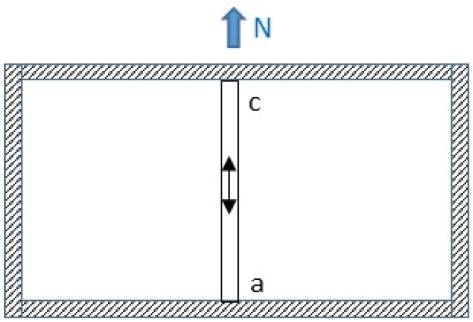
Therefore, the section cut locations to obtain the demand due to out-of-plane forces are:

- For out-of-plane moment, at about the center of each of the slab-wall interfaces; and, at a section about the center of the slab short span (main direction of moment transfer), as shown in Figure 8-6.
- For out-of-plane shear, at about the center of each of the slab-wall interfaces as shown in Figure 8-7.

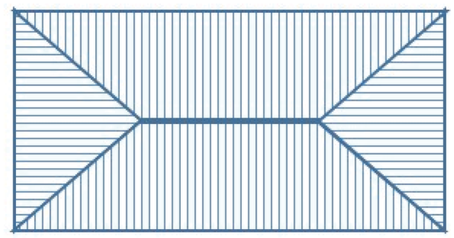
At these section cuts, the axial loads are also calculated to account for P-M and P-V interactions in the design calculations.

ACI 349 allows the calculation of the shear force at a distance equal to one slab effective depth from the face of the support, if the requirements specified in ACI 318 Section 11.1.3 are met. The FEA mesh may not include nodes at one effective depth away from the support and for such condition, the out-of-plane shear can be conservatively calculated at section cuts defined at the face of the support.

Figure 8-5 Types of out-of-plane moments in four-edged fixed slab

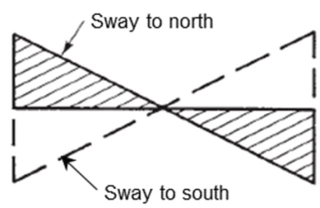


a) Slab showing middle strip (ac)



b) Distribution of gravity loads to supports

c) Moments due to gravity loads (strip ac)



d) Moments due to frame action (strip ac)

Figure 8-6 Section cut locations to determine out-of-plane moment demand due to gravity and frame action in X and Y directions

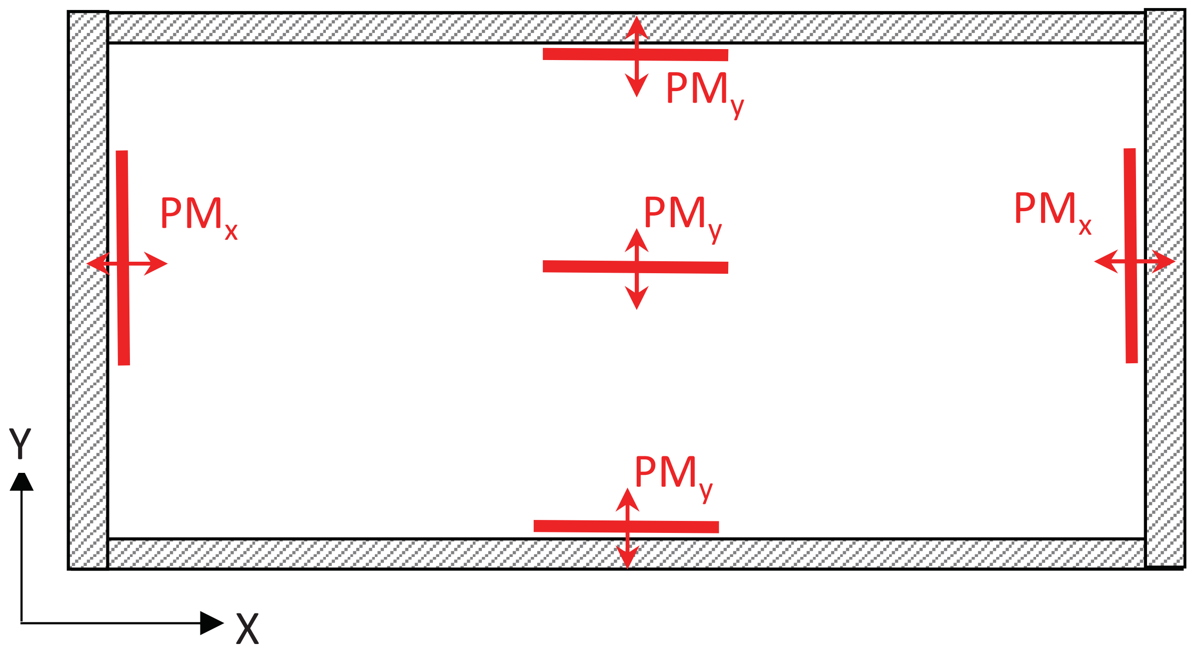
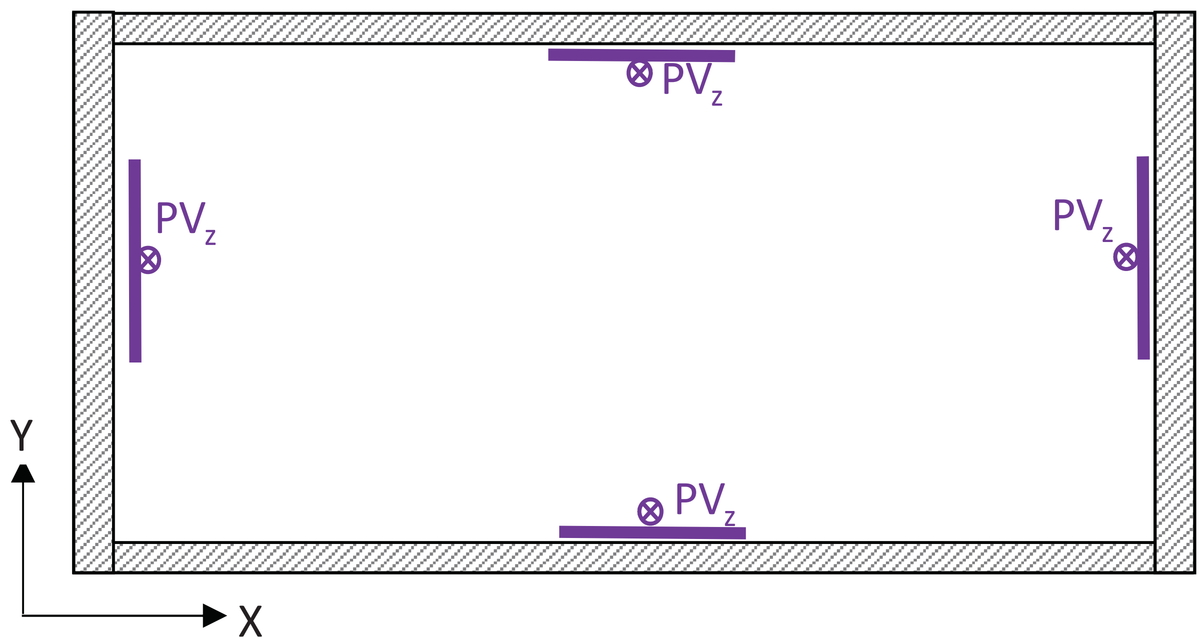


Figure 8-7 Section cut locations to determine out-of-plane shear demand due to gravity and frame action in X and Y directions



Due to seismic loads, structural walls may be subject to large overturning moments which are equilibrated by out-of-plane moment and out-of-plane shear in basemats at locations next to the wall ends (e.g., Figure 8-8). Therefore, section cuts are also defined at the ends of walls imposing overturning moments to basemats.

Considering a general case of a T-shaped wall shown in Figure 8-9, the basemat region at the wall end is subjected to two-way shear (punching shear). Thus, the out-of-plane shear demand in this region is calculated at the section cut shown in red in Figure 8-9. This critical section is defined by conservatively assuming that the axial load in the wall is concentrated in a width equal to the wall thickness, b_w. In agreement with ACI 318 Section 11.11.1.2, the critical section is obtained by adding half the basemat effective depth, *d*, at each side of the wall.

The basemat region outside the two-way shear region is subject to one-way shear. In this case, the out-of-plane shear demand is obtained at the section cuts shown in blue in Figure 8-9. In agreement with ACI 318 Section 11.1.3.1, the section cut is located at one basemat effective depth, *d*, from the face of the wall.

The section cut location for out-of-plane bending due to wall overturning is shown in blue in Figure 8-10. Based on recommendations in Reference 10.1.28, the section cut length is limited to three times the basemat thickness.

Figure 8-8 Out-of-plane forces in foundation basemat due to wall rocking

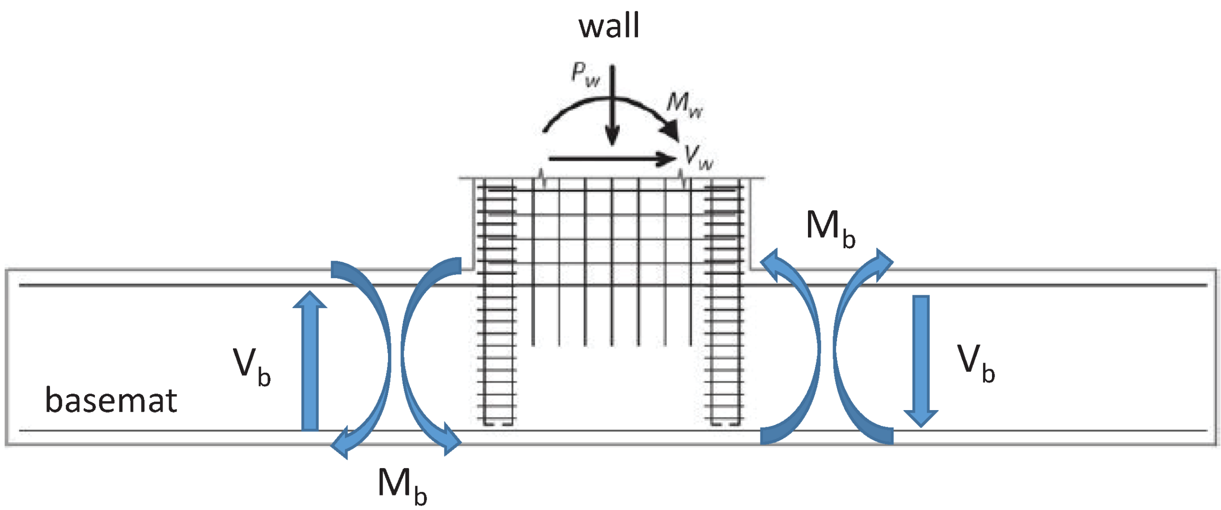


Figure 8-9 Section cuts for one-way and two-way shear in basemat subject to wall overturning

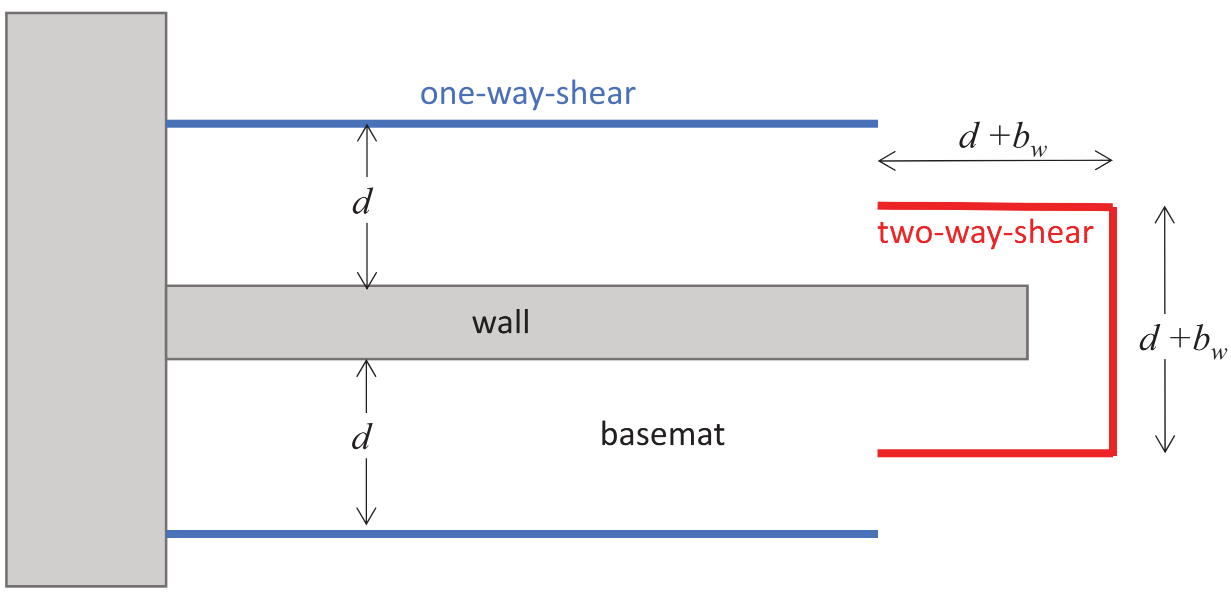
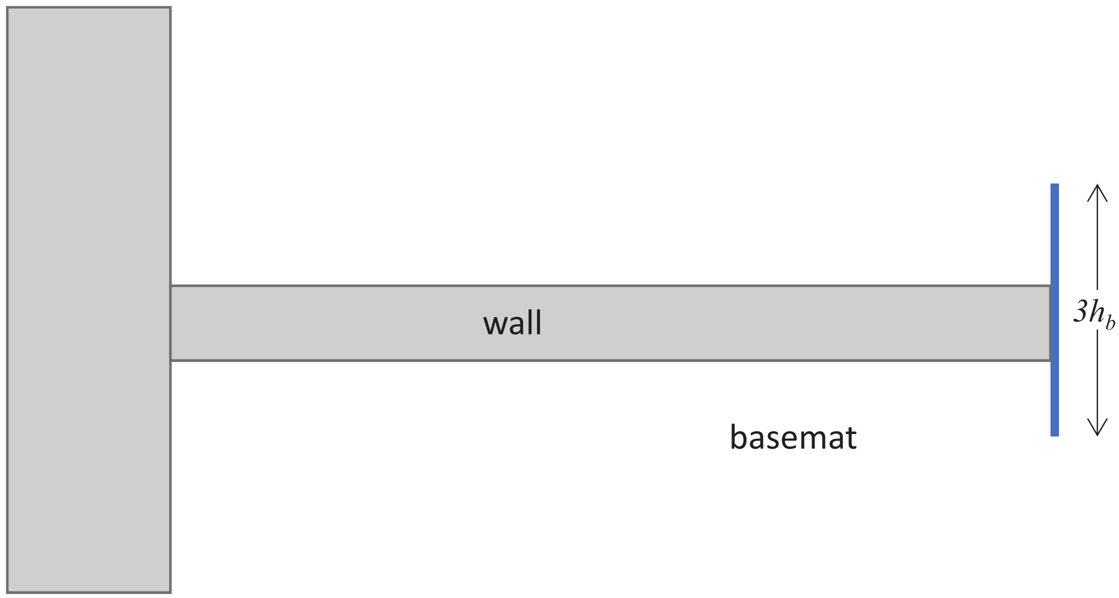


Figure 8-10 Section cuts for out-of-plane bending in basemat subject to wall overturning



8.5.2 Section Cuts for In-Plane (Diaphragm) Forces

In-plane forces include in-plane moments or chord forces, in-plane shear; and, shear transfer or collector forces.

Figure 8-11 shows a simple diaphragm supported by walls at its four edges and subject to seismic forces in the north-south direction which are represented by uniform load along the diaphragm length. In this simple configuration, the diaphragm can be idealized as a horizontal beam simply supported at the two ends, as shown in Figure 8-11c. In-plane moment and in-plane shear diagrams along the diaphragm length are also shown in Figure 8-11c for the uniformly distributed seismic load. These in-plane forces occur across the diaphragm depth (d)—the diaphragm depth is the diaphragm dimension parallel to the seismic forces and the diaphragm span is the distance between walls resisting the seismic forces.

As shown in Figure 8-11b, the in-plane moment is equilibrated by a compression and tension couple known as the chord force at the top and bottom edge of the diaphragm. From Figure 8-11c, the maximum chord forces occur at the mid-span of the diaphragm where the in-plane moment is the largest; on the other hand, the maximum in-plane shear occurs at the diaphragm supports (i.e., walls). Therefore, the section cut locations to obtain the in-plane moment and in-plane shear demand are:

- For chord forces, at about the mid-span of the diaphragm and close to diaphragm edges and oriented along the direction of the seismic force; and,

- For in-plane shear, at diaphragm support locations at walls and oriented along the direction of the seismic force.

These section cut locations are shown in Figure 8-12 and Figure 8-13 for seismic forces in north-south and east-west direction, respectively. At the section cut locations for chord forces, the out-of-plane moment is also calculated to evaluate the section design considering P-M interaction.

Figure 8-11 In-plane moment and shear at section cut ab, in a diaphragm supported by two walls at its ends (from Reference 10.1.21)

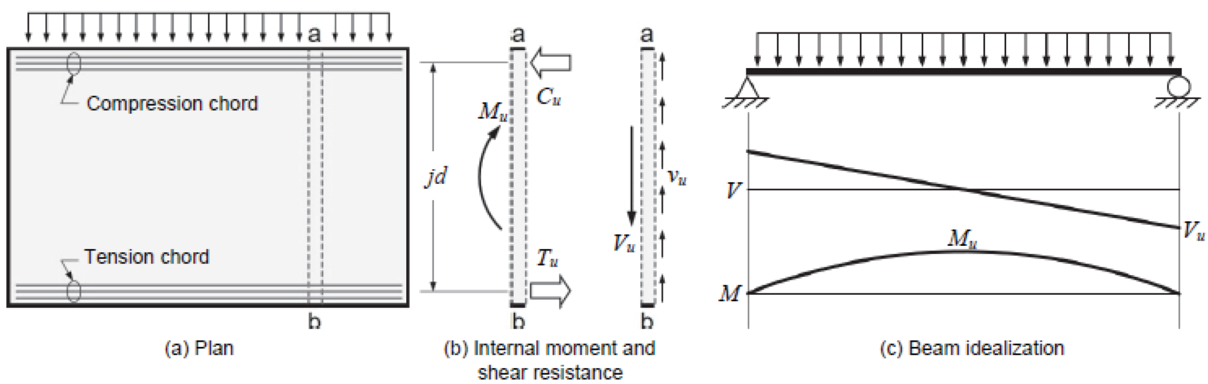


Figure 8-12 Section cut locations to determine in-plane shear and chord forces due to seismic force in Y direction

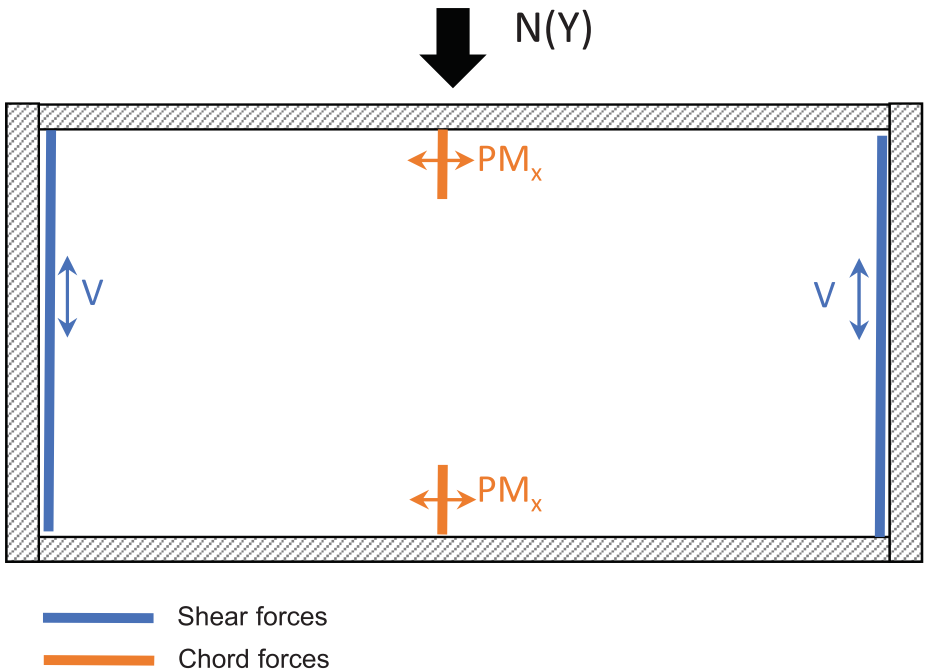
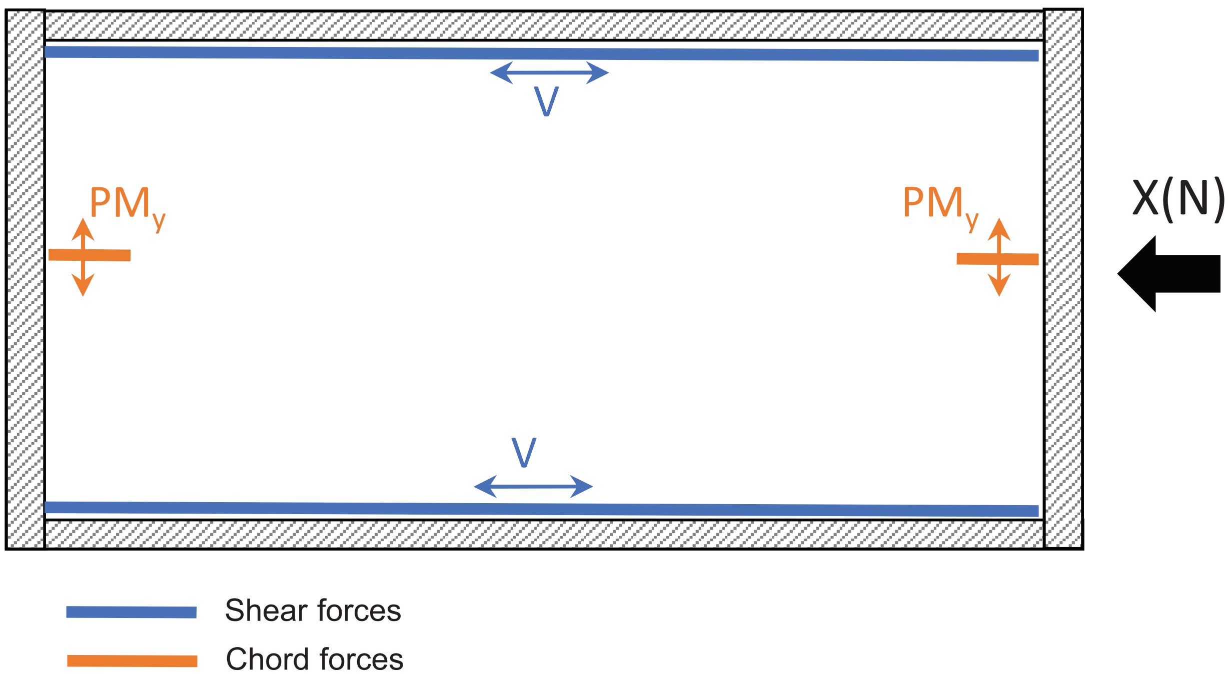


Figure 8-13 Section cut locations to determine in-plane shear and chord forces due to seismic force in X direction



In diaphragms where the supporting wall is not continuous across the diaphragm depth (Figure 8-14) a portion of the total diaphragm shear force, V_u, is transferred to the supporting wall through direct shear: $v_u l_{bc}$ (Figure 8-14b). The remaining portion is transferred through collector elements in terms of compression and tension forces. The maximum compression and tension force in the collectors occur at the wall ends (Figure 8-14b). Considering this load path, the collector forces at the ends of discontinuous walls are calculated at the section cut locations shown in Figure 8-15. At these section cuts, the out-of-plane moment is calculated to evaluate the section design considering P-M interaction.

Figure 8-14 Collector forces in a diaphragm (from Reference 10.1.21)

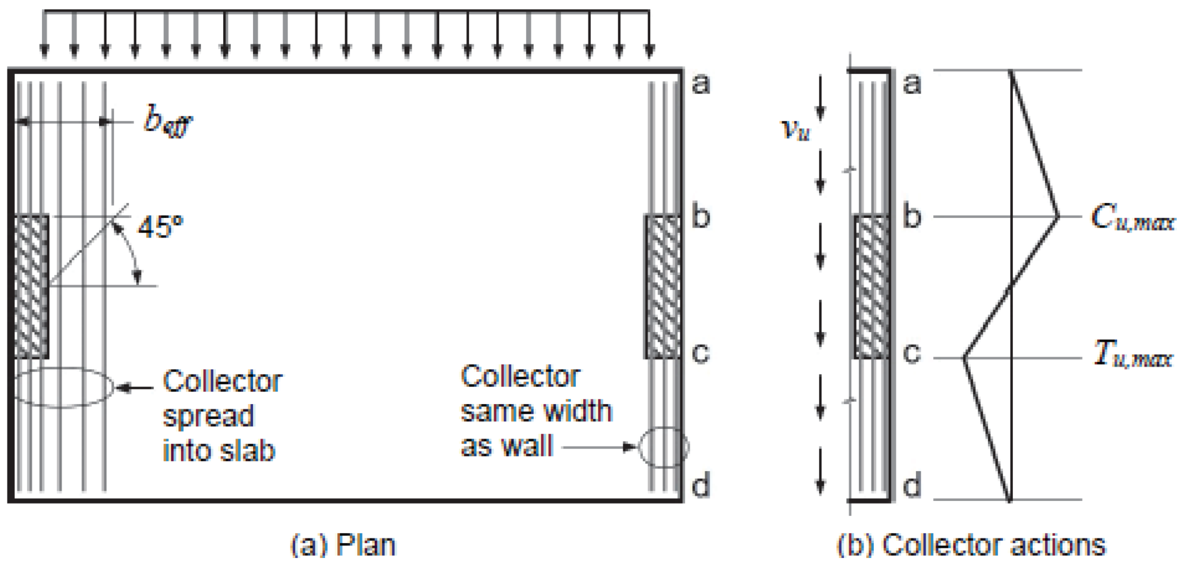
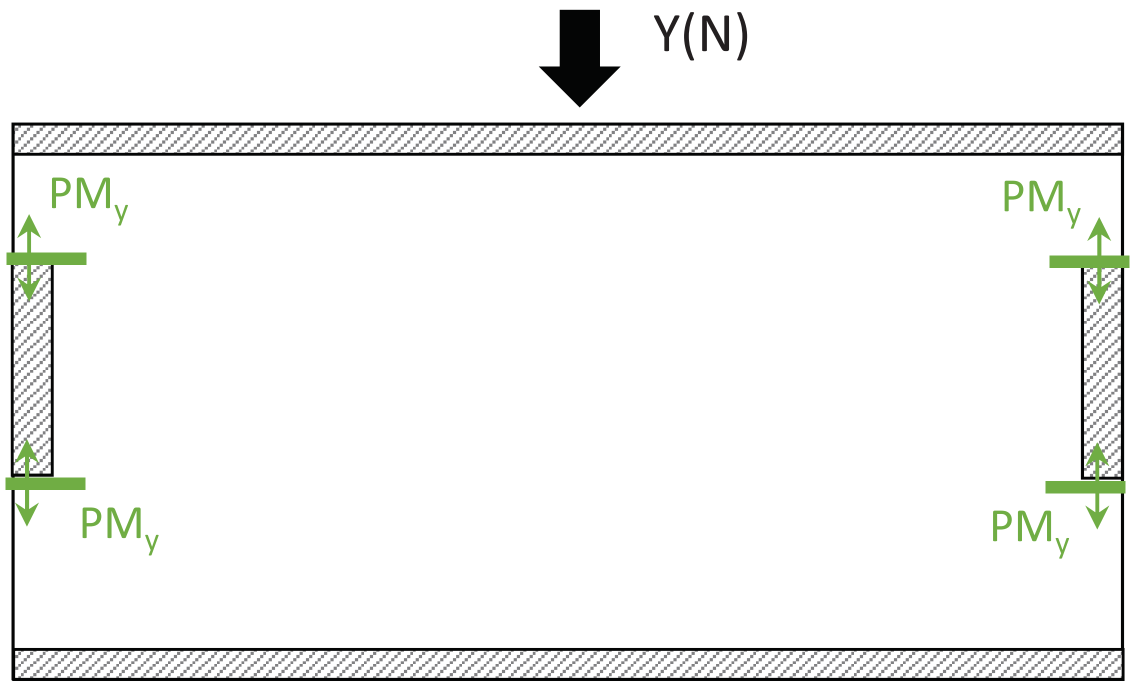


Figure 8-15 Section cut locations to determine collector forces due to seismic force in Y-direction



8.5.3 Section Cuts for Forces around Openings

Openings of any size are permitted by ACI 349 Section 13.4, provided that the total amount of reinforcement required for the structural member without the opening is maintained, shear requirements of ACI 349 Section 11.11.6 are satisfied, and amount of reinforcement interrupted is limited by location (e.g., inside or outside column strips). This is known as the ACI rebar area replacement rule.

Common design practice is to apply ACI rebar area replacement rule for small openings, on the order of one or two slab thicknesses (Reference 10.1.21). For larger openings, the slabs are designed to transfer the forces around the openings as explained in this section.

Figure 8-16 shows a sample slab with a large opening at the middle and subject to gravity load and seismic force along the north-south direction. With respect to gravity load and frame action, the direction of load transfer around the opening is indicated by the blue double-head arrows shown in Figure 8-16. These are also the directions of the out-of-plane moments around the opening. Therefore, the out-of-plane moment and out-of-plane shear around openings are calculated at section cut locations as shown in Figure 8-17. At these section cuts, the in-plane axial forces are also calculated such that design evaluations can be performed considering P-M and P-V interactions.

With respect to the in-plane forces, additional chord forces are generated due to in-plane bending of the small diaphragm segments on each side of the opening. For the diaphragm shown in Figure 8-16 subjected to seismic motion in the north-south direction, the small diaphragm segments are shown with yellow shaded areas on the north and south sides of the opening. A similar set of yellow shaded areas can be shown on the east and west sides of the opening to represent the small diaphragm segments that are active for seismic motion in the east-west direction.

Based on Figure 8-16 two types of chord forces are generated: primary chord forces as a result of in-plane bending of the slab as a whole; and secondary chord forces as result of in-plane bending of the diaphragm segments above and below the opening. As can be seen in Figure 8-17, the primary and secondary chord forces at the top and bottom slab edges are additive. Considering this global and local response behavior, the chord forces for the north diaphragm segment are calculated at the section cut locations shown in Figure 8-18 for seismic forces in the north-south direction. Similar sets of section cuts are considered for the south diaphragm segment and also for the east and west diaphragm segments for the seismic motion in the east-west direction. At these section cuts, the out-of-plane moment is also calculated to evaluate the section design considering P-M interaction.

Figure 8-16 Slab with a large opening at the middle and directions of forces and moment around the opening

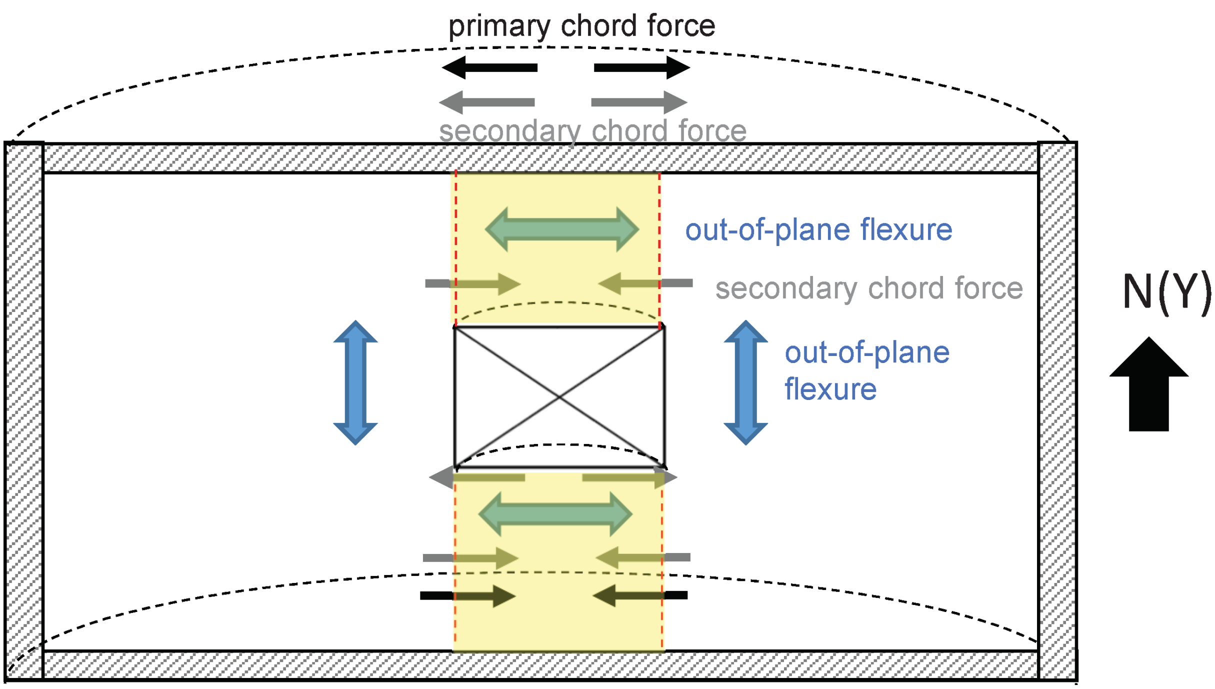


Figure 8-17 Section cut locations to determine out-of-plane moment and shear around openings

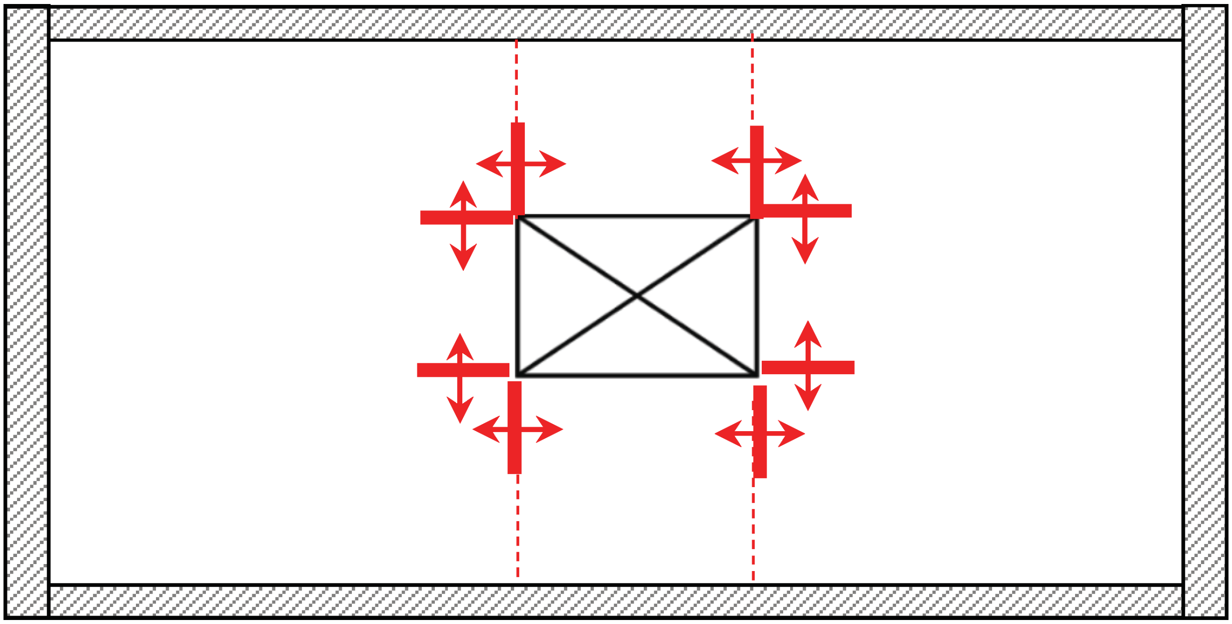
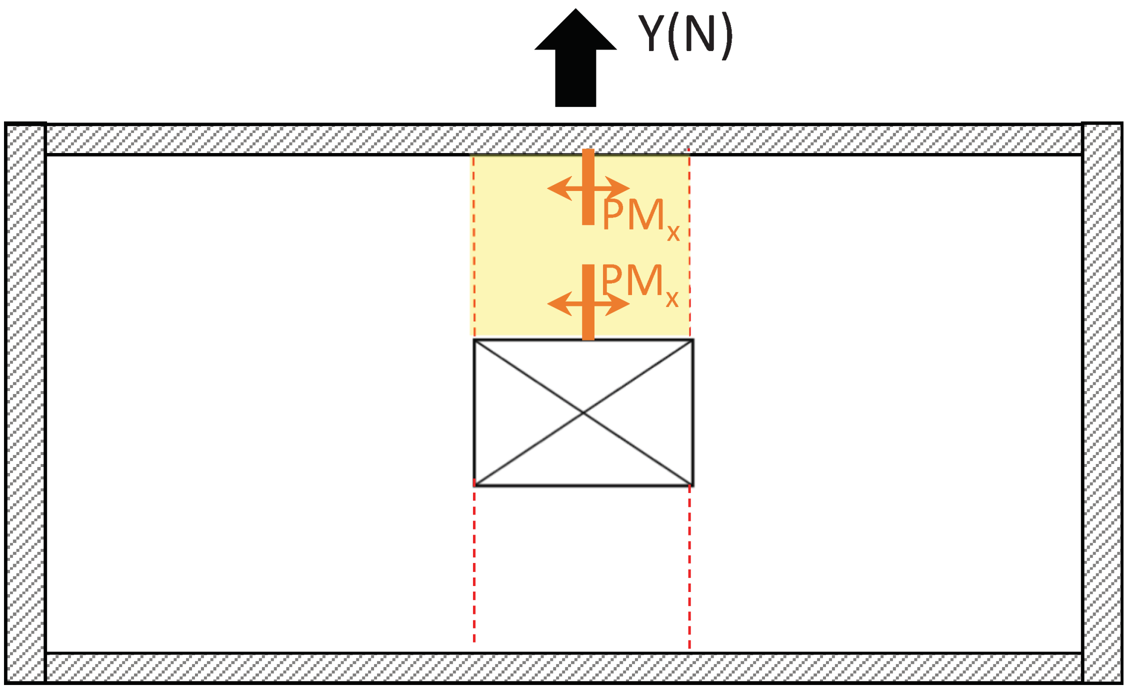


Figure 8-18 Section cut locations to determine chord forces in one diaphragm segment generated by an opening



8.5.4 Determination of Section Cut Locations using FEA Stress Resultants

The FEA stress resultants are shown in Figure 8-19 in the finite element local coordinate system, in which:

- N_{11} and N_{22}, are the membrane axial forces per unit length
- N_{12}, is the membrane shear force per unit length
- M_{11} and M_{22}, are the out-of-plane moments per unit length
- M_{12}, is the twisting moment per unit length; and,
- Q_{13} and Q_{23}, are the out of plane shear forces per unit length

The element local coordinate system is aligned with the main directions of the reinforcement in slabs and basemat, as shown in Figure 8-20.

According to the local coordinate system shown in Figure 8-19, the positive moments, M_{11} and M_{22}, cause tension in the top slab reinforcement. This is contrary to the sign convention used by ACI in which a positive moment results in tension at the bottom reinforcement. The ACI sign convention is used whenever the code requirements refer to the moment signs.

The FEA stress resultants are calculated for each load combination used in the design process. For load combinations that include seismic forces, two sets of FEA stress resultants are obtained: one set due to the non-seismic loads included in the load combination (S_{ns} in Equation 8-1) and one set due to the seismic loads (S_s in Equation 8-1).

For the purpose of showing the critical locations for design, the peak FEA stress resultants due to the seismic loads are obtained. Peak FEA stress resultants can be calculated through either response spectrum analysis or time history analysis and considers the effect of input motion in three directions.

If time history analysis is used, a representative input motion set is used. FEA stress resultant time histories are calculated for each input motion direction and then combined using algebraic summation. For each finite element, the absolute peak responses are determined from the combined stress resultant time histories.

Considering that seismic motion can occur in either positive or negative direction, the FEA stress resultant sets due to non-seismic and seismic loads are combined as shown below:

$$\begin{aligned} S_{r1} &= S_{ns} + S_s \\ S_{r2} &= S_{ns} - S_s \end{aligned} \qquad \text{Equation 8-1}$$

Where:

S_{ns} are the stress resultant set due to non-seismic loads; and,

S_s are the peak stress resultant set due to seismic loads (stress resultant peaks from seismic analyses are taken as positive values)

Using the combined stress resultant sets S_{r1} and S_{r2}, contour plots are generated showing the peak responses at each finite element in the FEA model of the slab or basemat.

The section cut locations and their lengths are determined as follows:

- For each demand type generate the corresponding FEA stress resultant contour plot based on Table 8-1. For in-plane shear, V, use the contour plot corresponding to N_{12}. For chord and collector forces, the demand type of interest is the axial load, P, thus, the contour plot corresponding to N_{11} and N_{22} is used. For out-of-plane moment, M, use the contour plot corresponding to M_{11} and M_{22}. For out-of-plane shear, V_z, use the contour plot corresponding to Q_{13} and Q_{23}.

- Using each contour plot, identify the regions with peak responses. Define section cuts at these locations. Define the length of section cuts based on the size of the regions with peak responses. The length of section cuts need not be less than three times the slab thickness unless the stress resultant changes sign along the cut or it is limited by openings.
- Additional section cuts are defined to calculate P-M and P-V_z demands if significant tension is present. These section cuts are considered in addition to the section cuts defined in Sections 8.5.1, 8.5.2, and 8.5.3 for the evaluation out-of-plane moment and out-of-pane shear demands.

Figure 8-19 Finite element stress resultants in their local coordinate system (Reference 10.1.8)

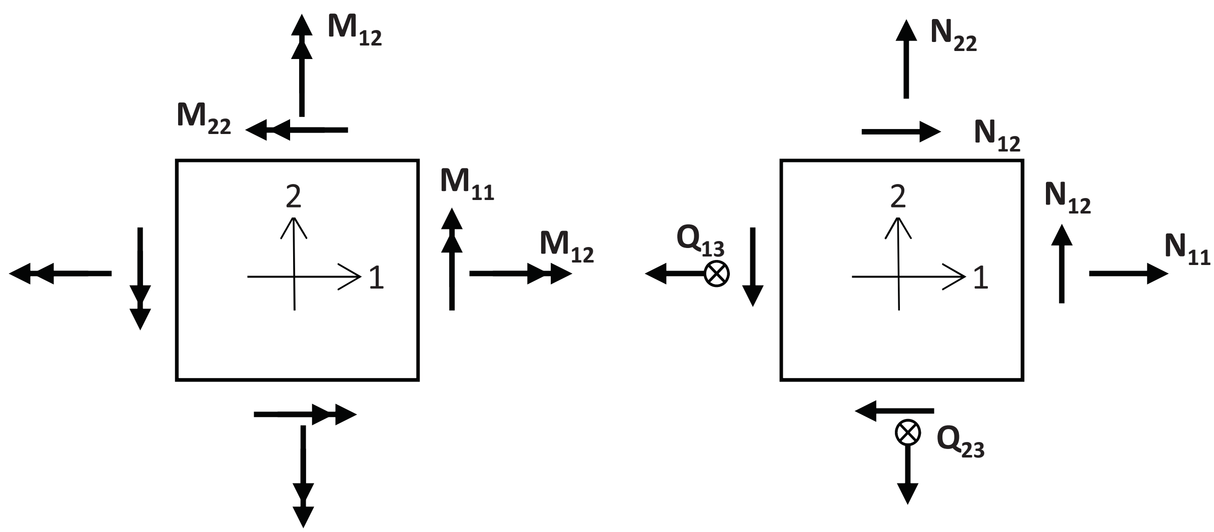
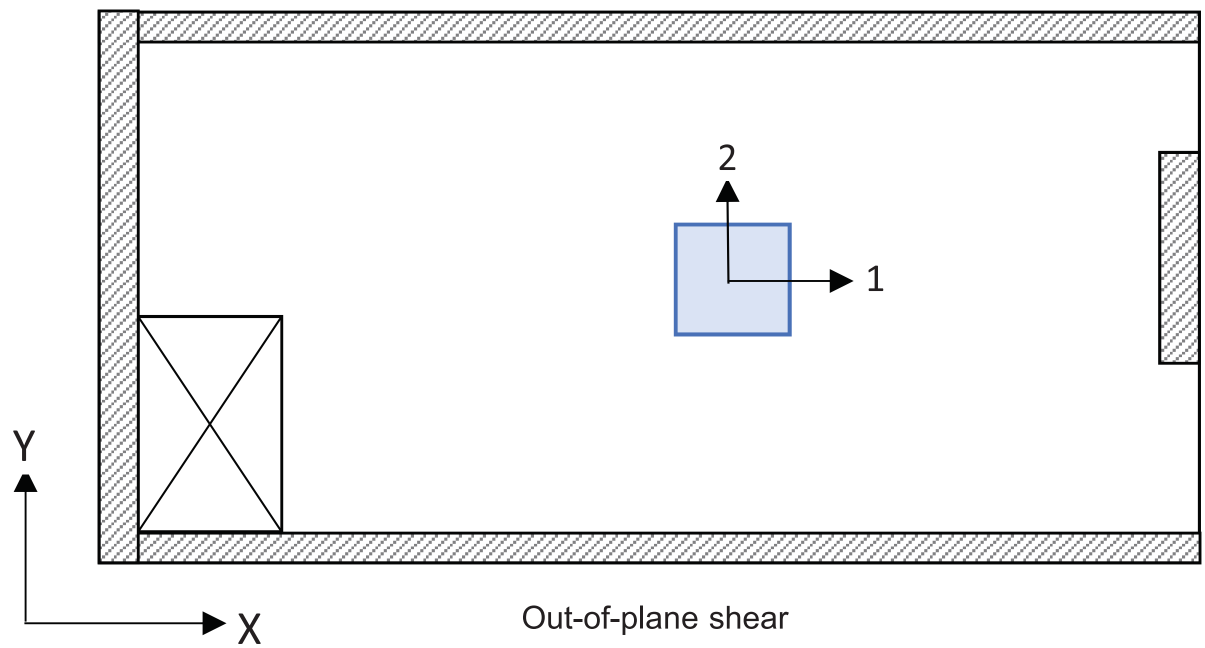


Table 8-1 Demand types in floor slab and basemat and corresponding FEA stress resultant

Demand Type	Corresponding FEA stress resultant[1]
In-plane shear, V	N_{12}
Chord force and collector force, P	N_{11}, N_{22}
Out-of-plane moment, M	M_{11}, M_{22}
Out-of-plane shear, V_z	Q_{13}, Q_{23}

[1] Finite element forces and moments per unit length (Figure 8-19)

Figure 8-20 Typical slab showing FEA local coordinate system (at the center) aligned with the orientation of the primary reinforcement running in the X- and Y-directions



8.6 Required Strength for T-Beam Design

Similar to slabs, the largest strong-axis moment in beams are expected at the supports; that is, at the beam ends. Due to gravity loads, there is also a possibility of developing significant moment around the beam mid-span. Therefore, the beam demand is calculated at section cuts at the following locations: 1) at the beam ends, 2) at mid length; and, 3) at quarter points along the length.

The section cuts include the effective slab width. In reference to Figure 8-4, the following force resultants are used for T-beam design:

- the strong-axis bending and shear, M3 and V2; and,
- the axial load P.

The design considers the interaction of axial load with moment and shear; i.e. P-M and P-V interactions.

8.7 Required Strength for Column Design

There are no significant loads applied along the height of columns and, hence, the maximum design forces and moments are expected at the column ends and are calculated from section cuts defined at these locations.

In reference to Figure 8-3, the following force resultants are used for column design:

- Bending moments M2 and M3;
- axial force P; and,
- shear forces V2 and V3.

Columns are designed for biaxial bending unless one of the bending moments is relatively small. In this case, columns are designed for uniaxial bending; i.e., using P-M interactions for the strong axis and checked for strength in the orthogonal direction (Reference 10.1.23).

8.8 Enveloping P-M Demand

The member forces for the safe shutdown earthquake, E_{ss}, are obtained from seismic analysis of the Seismic Category (SC) I and II concrete structures and the calculated force and moment demands are time-histories. The demand due to non-seismic loads in load combinations (9-6) and (9-9) are added to the E_{ss} demands at each time step. Thus, the required strengths calculated for the load combinations including E_{ss} are time-histories.

For a given section cut, the axial force and moment (P-M) interaction is considered at each time step for the full duration of time-history. To reduce the P-M evaluation points, a procedure is applied to extract the demand envelope from P-M time-history data. In this approach, demand-to-capacity ratios only need to be calculated for the set of points on the demand envelope curve.

8.9 Load Combinations

The concrete members are designed to have required strengths at least equal to the effects of factored load combinations shown below. These load combinations are obtained from Section 9.2.1 of ACI 349 and expanded to include extreme snow load condition. The symbols are defined in Table 1-3.

Per ACI 349, Section 9.2.2, the structural effects of differential settlement, creep, and shrinkage are either included as part of dead load D in ACI 349 Equations 9-4 through 9-9 or a separate engineering evaluation is performed to address their effects.

Per ACI 349, Section 9.2.3, load combinations (9-1) through (9-9) are evaluated with 0.9D to assess the adverse effects of reduced dead load. For any other load (e.g., L), if the load reduces the effects of other loads, the corresponding factor for that load is taken as 0.9 of the assigned factor, if it can be demonstrated that the load is always present. Otherwise, the factor is taken as zero.

ACI 349 Section 9.2.10 allows for a reduction in the load effects of the seismic load (E_{ss}) in (9-6) and (9-9) by 10 percent. In this methodology, the seismic load effects are not reduced.

Table 8-2 Load combinations for concrete structures

Load Combination	ACI 349 Equation
Normal Load Combinations	
$1.4(D + F + R_o) + T_o$	(9-1)
$1.2(D + F + T_o + R_o) + 1.6(L + H) + 1.4C_{cr} + 0.5(L_r \text{ or } S \text{ or } R)$	(9-2)
$1.2(D + F + R_o) + 0.8(L + H) + 1.4C_{cr} + 1.6(L_r \text{ or } S \text{ or } R)$	(9-3)
Severe Environmental Load Combinations	
$1.2(D + F + R_o) + 1.6(L + H + E_o)$	(9-4)
$1.2(D + F + R_o) + 1.6(L + H + W)$	(9-5)
Extreme Environmental Load Combinations	
$D + F + 0.8L + C_{cr} + H + T_o + R_o + E_{ss}$	(9-6)
$D + F + 0.8L + H + T_o + R_o + (W_t \text{ or } W_h)$	(9-7)
$D + F + 0.8L + H + T_o + R_o + S_e$ (see Note 1)	
Abnormal Load Combinations	
$D + F + 0.8L + C_{cr} + H + (T_a + R_a + 1.2P_a)$	(9-8)
$D + F + 0.8L + H + (T_a + R_a + Pa) + (Y_r + Y_j + Y_m) + E_{ss}$	(9-9)

Note 1: this load combination corresponds to extreme snow load.

8.10 Basic Design Requirements

8.10.1 Materials

Consistent with ACI 349, the basic material properties are as follows:

- Normal weight concrete with a minimum compressive strength, $f_c^{'}$, of 5,000 psi.
- Reinforcing steel resisting flexural and axial forces consist of deformed bars conforming to ASTM A706, Grade 60, yield strength f_y = 60,000 psi. Other reinforcing steel can consist of deformed bars conforming to either ASTM A706 or ASTM 615, Grade 60.
- Shear reinforcement is in the form of stirrups or ties with f_{yt} = 60,000 psi.

In order to ensure proper encasement of reinforcement and to minimize honeycombing, the nominal maximum size of coarse aggregate is designed to not be larger than the values specified in ACI 318 Section 3.3.2.

8.10.2 Limits on Flexural Reinforcement

8.10.2.1 Slabs and Basemat

Slab and basemat sections have ratio of reinforcement area to gross concrete area at least equal to the reinforcement ratio for shrinkage and temperature specified in ACI 349 Section 7.12.2.1 for members with thickness less than 48 in., and in ACI 349 Section 7.12.2.2 for members with thickness of 48 in. or more. Table 8-3 shows the minimum reinforcement ratio for shrinkage and temperature for typical slab and basemat thicknesses. Additionally, slab and basemat sections subjected to flexure shall have a ratio of reinforcement area at the tension face to gross concrete area of at least 0.0018 (ACI 349 Section 7.12.4 and RG 1.142).

The minimum reinforcement is provided in each direction. Reinforcement needed to resist design loads can be included as part of the minimum reinforcement.

Table 8-3 Minimum reinforcement ratios for shrinkage and temperature

Member Thickness (in.)	ρ min[1]
24	{{
36	
60	
96	}}2(a),(c)

[1] For members having thickness less than 48 in., ρ_{min} as per ACI 349 Section 7.12.2.1, using Grade 60 deformed bars. For members having thickness of 48 in. or more, ρ_{min} is the total minimum reinforcement ratio calculated per ACI 349 Section 7.12.2.2, considering both faces, and using No. 11 interior bar spaced at 12 in.

[2] Minimum reinforcement ratio of A/100 controls for this member thickness. A is calculated to reflect the shaded area of Figure 8-21. Minimum reinforcement is 2*(A/100)/(12"*member thickness)". The factor 2 reflects reinforcement of the top and bottom layers of reinforcement in the section.

Per ACI 349 Section 7.12.2.2, for members having a thickness of 48 in. or more, minimum reinforcement is provided on each face based on Equation 8-2.

$$A'_{s,min} = \frac{f_t' A}{f_s''} \le \frac{A}{100} \qquad \text{Equation 8-2}$$

where

A is the effective tensile area of concrete surrounding the reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars (in.^{2}). ACI 329 Section R2.1 defines the tensile strength of concrete f'_t as f_{ct}. ACI 318-18 Section R8.6 defines f_{ct} as per equation below:

$$f_t' = f_{ct} = 6.7\sqrt{f_c'}$$

$$f_s'' = 0.6f_y$$

The minimum reinforcement ratio shown in Table 8-3, for members with thickness of 48 in. or more, is calculated for the typical case of No. 11 interior bars (on top of transversal bars) spaced at 12 in. on center and with 2 in. of concrete cover, as shown in Figure 8-21. The concrete strength (f_c') is assumed as 5,000 psi. For larger bars, reinforcement in multiple layers, or different f_c', the minimum reinforcement is to be calculated based on Equation 8-2.

Figure 8-21 Typical slab or basemat section with No.11 interior bar at 12 in. for the calculation of minimum reinforcement based on ACI 349 Section 7.12.2.2

{{

}}2(a),(c)

The arrangement of bars within concrete members must allow for sufficient concrete on the sides of each bar to transfer forces into or out of the bars. In this regard, the spacing limits for reinforcement are as specified in ACI 318 Section 7.6.

Based on the requirements specified in ACI 318 Section 7.6 and the recommendations of ACI 421.3R (Reference 10.1.20), the default bar spacing for slabs is 12 inches on center, each way and each face.

Depending on the area of steel needed at a specific slab section, the reinforcement spacing can be reduced by 9 inches. If additional reinforcement is needed, this is placed in additional layers directly above bars in the bottom layer and with 1 inch minimum spacing between them.

Additional requirements of reinforcement resisting in-plane diaphragm forces are included in ACI 349 Section 21.11.7. Reinforcement in the foundation basemat is satisfied by the requirements in ACI 349 Section 21.12.2.

8.10.2.2 T-Beams

The minimum reinforcement for T-beams is to not be less than that given by ACI 318 Eq. 10-3 below

$$A_{s,min} = \frac{3\sqrt{f_c'}b_w d}{f_y} \geq 200 b_w d / f_y \qquad \text{Equation 8-3}$$

Where *d* (in.) is the distance from the extreme compression fiber to the centroid of the rebar in tension, b_w (in.) is the width of the web, except that b_w is replaced by the smaller of $2b_w$ or the width of the flange in tension for beams with flanges.

ACI 349 Equation 10-4 calculates, *s*, the maximum spacing of reinforcement closest to the tension face as a function of stress, f_s, in reinforcement closest to the tension face at service load and c_c, the least distance from surface of reinforcement to the tension face. From ACI 349 Section 10.6.4, the stress in the reinforcing steel under service loads, f_s, is permitted to be taken as 40 percent of f_y. The least distance, c_c, can be conservatively taken as 2 in. Substitution of this information into ACI 349 Equation 10-4 results in *s* of 20 in. This is equal to the maximum spacing of 12*(40,000/f_s) in Section 10.6.4 of ACI 349.

Also, for beams with depths exceeding 36 inches, longitudinal skin reinforcement is required along both faces of the member, as specified in ACI 318 Section 10.6.7. The minimum bar spacing is determined based on the requirements of ACI 318 Section 7.6.

8.10.2.3 Columns

The minimum and maximum areas of longitudinal reinforcement, as provided in ACI 349 Section 10.9.1, are:

$$A_{st,min} \geq 0.01 A_g \qquad \text{Equation 8-4}$$

$$A_{st,max} \leq 0.08 A_g \qquad \text{Equation 8-5}$$

where A_g is the gross cross-sectional area of the member.

The minimum bar spacing is determined based on the requirements of ACI 318 Section 7.6 for compression members.

8.10.3 Minimum Concrete Cover

The minimum clear cover for the foundation basemats is three inches. This corresponds to concrete exposed to earth (ACI 318 Section 7.7.1).

For the floor slabs, the minimum clear cover is shown in Table 8-4. This is based on ACI 318 Section 7.7.1 and, for slabs thicker than 12 inches, the specified clear covers facilitate the placement of up to 1½-inch thick embedment plates with a minimum of 1 bar diameter or 1 inch clear distance from the reinforcement to the embedded items, in agreement with ACI 117-10, Section 2.3.1 (Reference 10.1.22).

For beams and columns, the minimum clear cover is 1-1/2 in.

Table 8-4 Minimum specified clear cover for slabs

Member Effective Depth[1]	Slab Thickness[1,2]	Specified Cover[3]	Tolerance on d[4]
{{			
			}}[2(a),(c)]

[1] Exclude non-structural liners, pool liners, metal decking, and ribs.

[2] Correlation of member thickness to effective depth depends on clear cover, bar size, bar orientation and layer, and number of layers. This table is approximate and assumes members have only one curtain of reinforcement each face with maximum No. 11 bar size.

[3] The maximum bar size to be used in the NuScale design is No. 11 for which only 3/4" concrete cover is required per Section 7.7.1 of ACI 318. The specified cover in the table takes into account embedded plate thickness as follows:
For effective depth ≤ 8" and for slab thickness <12", no embedded plate is needed.
For member effective depth between 8" and 24" and for slab thickness between 12" and 28", specified cover takes into account 1" thick embed plates with a maximum bar size of No. 8 to satisfy a clear distance of 1" from the reinforcement to the embed plate, in agreement with ACI 117-10, Section 2.3.1.
For member effective depth >24" and for slab thickness larger than 28", specified cover takes into account up to 1-1/2" thick plate with a maximum bar size of No. 11. A larger cover is needed for thicker embed plates.

[4] The tolerance requirements are in agreement with the requirements of Section 7.5.2.1 of ACI 349 where 3/8" tolerance is required for member effective depth ≤ 8" and 1/2" tolerance is required for member effective thickness > 8".

8.10.4 Out-of-plane Shear Reinforcement for Beams and Slabs

Based on ACI 349 Section 11.4.1, shear reinforcement consists of stirrups perpendicular to axis of member and spirals, circular ties, or hoops.

Typical stirrups details for beams and their anchorage requirements are included in ACI 318 Sections 12.13.2 and 12.13.5.

For beams with torsion or compression reinforcements, ACI 318 Sections 7.13 and 11.5.4.1 include additional requirements.

Code requirements for standard hooks are included in ACI 318 Sections 7.1 and 7.2.1.

For two-way slabs, i.e., slabs with longest span to shortest span ratio no larger than two, ACI 349 Section 11.11.3 allows the use of single-leg, multiple-leg, and closed stirrups, provided there are longitudinal bars in the corners of the stirrups. Stirrups details for two-way shear are shown in ACI 318 Figure R11.11.3.

8.10.5 Lateral Reinforcement for Compression Members

Lateral reinforcement for columns consists of spirals or ties. The requirements for spirals and ties are shown in ACI 349 Sections 7.10.4 and 7.10.5, respectively.

Ties are provided in compression members for the following reasons:

- Ties restrain the longitudinal bars from buckling out through the surface.
- Properly detailed ties confine the concrete core, providing increased ductility.
- Ties serve as shear reinforcement.

ACI 349 Section 7.11 species compression reinforcement in beams be enclosed by ties or stirrups satisfying the size and spacing requirements for compression members as discussed below.

The bars are enclosed by lateral ties, at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for larger and bundled bars.

The maximum vertical spacing of ties is given by ACI 349-13, Section 7.10.5.2 as:

$$
\begin{aligned}
s_{max} \quad &\leq 16 \text{ x longitudinal bar diameter} \\
&\leq 48 \text{ x tie bar diameter} \\
&\leq \text{least dimension of the member}
\end{aligned}
$$

ACI 349 Section 7.10.5.3 outlines the arrangement of ties in a column cross section as illustrated in ACI 349-13, Figure R7.10.5.

ACI 349 Section 7.10.5.4 requires the bottom and top ties be placed not more than one-half a tie spacing above or below the slab, respectively.

ACI 349 Section 7.9.1 requires bar anchorages in connections of beams and columns be enclosed by ties, spirals, or stirrups. Generally, ties are most suitable for this purpose and are arranged in agreement with ACI 349-13, Section 7.10.5.4.

8.10.6 Bundled Bars for Beams and Columns

In agreement with ACI 349 Section 7.6.6, groups of parallel reinforcing bars bundled in contact to act as a unit are limited to four in any one bundle.

Bundled bars are enclosed within stirrups or ties. Bars larger than No. 11 are avoided in a bundle.

Where spacing limitations and minimum concrete cover are based on a bar diameter, a unit of bundled bars is treated as a single bar of a diameter d_{be}, derived from the equivalent total bar area (Reference 10.1.23), A_s, as $d_{be} = \sqrt{\frac{4A_s}{\pi}}$.

8.10.7 Development of Deformed Bars

According to ACI 349 Chapter 12, extensions of reinforcement must be provided beyond the locations of peak stress in the reinforcement in order to fully develop the bars. These locations of peak stress are referred to as critical sections, and they occur at points of maximum stress (e.g., maximum bending moment) and at locations where adjacent reinforcement is terminated. The reinforcement is developed on both sides of a critical section by means of embedment length, hook, or mechanical device, although hooks are not used to develop bars in compression.

8.10.7.1 Development of Deformed Bars in Tension

The development length l_d (in.) in terms of bar diameter d_b (in.) for bars in tension is as shown in ACI 318 Section 12.2.2, where λ is 1 (i.e., normal weight concrete). The development length l_d is no less than 12 in.

For No. 7 or larger uncoated bars with f_y = 60,000 psi, the development length in tension is obtained by Equation 8-6, for structural members satisfying the spacing and cover requirements of ACI 318 Section 12.2.2 listed below:

- Clear spacing of bars being developed or spliced not less than d_b,
- Clear cover not less than d_b; and,
- Stirrups or ties throughout l_d not less than the code minimum

$$l_d = \left(\frac{60,000\Psi_t}{20\sqrt{f_c'}}\right)d_b \qquad \text{Equation 8-6}$$

Table 8-5 shows the development length in terms of d_b for different concrete members with varying concrete strengths. Reduction in l_d is permitted where reinforcement in a flexural member is in excess of that required by analysis as specified in ACI 349 Section 12.2.5.

Table 8-5 Development lengths of No. 7 and larger uncoated Grade 60 bars satisfying spacing and cover requirements of ACI 318 Section 12.2.2

f_c' (psi)	l_d (in.)	
	Top rebar in horizontal members	Rebar and other members
5000	{{	
6000		
7000		}}2(a),(c)

8.10.7.2 Development of Deformed Bars in Compression

Based on ACI 349 Section 12.3, the development length for deformed bars in compression is

$$l_{dc} = 18d_b \qquad \text{Equation 8-7}$$

8.10.7.3 Development of Bundled Bars

Development length of individual bars within a bundle, in tension or compression, is that for the individual bar, increased 20 percent for three-bar bundle, and 33 percent for four-bar bundle.

8.10.7.4 Development of Standard Hooks in Tension

Development length of deformed bars in tension terminating in a standard hook is determined based on the requirements of ACI 318 Section 12.5. Hooks are not considered effective in developing bars in compression.

8.10.7.5 Splices of Reinforcement

ACI 349 Section 12.14 allows splicing of reinforcement by means of lap, mechanical, and welded splices. General requirements are included in ACI 349 Section 12.14.

Splices of deformed bars in tension and compression are covered in ACI 349 Sections 12.15 and 12.16, respectively. Splices in tension use a Class B splice.

In general, the minimum length of lap for tension lap splices is $1.3l_d$, as required for a Class B splice, where l_d is calculated. The minimum length of lap for compression lap splices is $30d_b$.

Specific requirement for splices in columns are included in ACI 349 Section 12.17.

8.10.7.6 Control of Deflections for Beams and Floor Slabs

In accordance with ACI 349 Section 9.5.1, RC members subjected to flexure have adequate stiffness to limit deflections or deformations that adversely affect the strength or serviceability of structural and nonstructural elements. Unless computation of deflections indicates a lesser thickness can be used, the minimum thickness specified herein applies.

The minimum thickness for beams or one-way slabs, i.e., slabs with longer to shorter span ratio larger than two, are included in ACI 349 Table 9.5(b).

For two-way slabs, the minimum thickness in ACI 349 Table 9.5(c) applies.

8.11 Member Capacities

8.11.1 One-Way Out-of-Plane Shear Capacity of Slabs and Basemat

The nominal out-of-plane shear strength, V_n, is calculated for one foot of member section. Based on ACI 349 Equation 11-2:

$$V_n = V_c + V_s \qquad \text{Equation 8-8}$$

where V_c is the nominal shear strength provided by concrete and V_s is the nominal shear strength provided by shear reinforcement. The shear strength is obtained by multiplying V_n by ϕ = 0.75.

The axial load acting at the section is considered in the calculation of shear strength. The shear strength considering the axial load interaction is calculated using ACI 349 Equations 11-4 and 11-8, shown below:

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}b_w d, \quad N_u \geq 0 \textit{ (compression)} \qquad \text{Equation 8-9}$$

$$V_c = 2\left(1 + \frac{N_u}{500A_g}\right)\sqrt{f_c'}b_w d \geq 0, \quad \mathrm{N_u} < 0 \textit{ (tension)} \qquad \text{Equation 8-10}$$

where N_u/A_g (psi) is the axial stress acting on the section, b_w (in.) is the width of the section (12 in. = 1 ft), and d (in.) is the distance from the compression face to the centroid of the flexural reinforcement in tension. The factored axial force, N_u (lb.), is positive in compression and negative in tension. At an axial tensile stress larger than 500 psi, the nominal shear strength provided by concrete becomes zero.

The nominal shear strength provided by shear reinforcement, in the way of shear ties in slabs and basemat, is obtained using ACI 349 Equation 11-15, shown below:

$$V_s = \frac{A_v f_{yt} d}{s} \le 8\sqrt{f_c'} b_w d \qquad \text{Equation 8-11}$$

where A_v (in.^{2}) is the area of shear reinforcement in width b_w, f_{yt} (psi) is the yield stress of the shear reinforcement, and s (in.) is the spacing of the shear reinforcement, measured perpendicular to the shear section cut. An example illustrating the application of A_v, b_w and s for two slab or basemat section cuts is shown in Figure 8-22.

The spacing s of the shear reinforcement does not exceed $d/2$ nor 24 in. (ACI 349 Section 11.4.5.1). For the case when $V_s > 4\sqrt{f_c'} b_w d$, the maximum spacing is reduced by one-half (ACI 349 Section 11.4.5.3).

Figure 8-22 Out-of-plane shear reinforcing example

{{

}}2(a),(c)

According to ACI 349 Section 11.4.6, minimum shear reinforcement is not required for slabs and basemats.

8.11.2 Punching Shear in Slabs and Basemat

The shear capacity of concrete when subject to concentrated loads or around columns in flat slabs is the smallest of the following equations (ACI 349 Section 11.11.2.1):

$$V_c = \left(2 + \frac{4}{\beta}\right)\sqrt{f_c'}\, b_o d$$ Equation 8-12

$$V_c = \left(2 + \frac{\alpha_s d}{\beta}\right)\sqrt{f_c'}\, b_o d$$ Equation 8-13

$$V_c = 4\sqrt{f_c'}\, b_o d$$ Equation 8-14

where b_o is the critical shear perimeter around the concentrated load area or column for a center and edge column, β is the ratio of long side to short side of the loaded area and d is the effective depth of the member; α_s is 40 for interior loads or columns, 30 for loads or columns on the edge of the slab, and 20 for loads or columns on the slab corners.

For columns or concentrated loads close to the slab edge, the critical perimeter can be obtained following the procedure shown in MacGregor, (Reference 10.1.17).

Figure 8-23 Typical locations of critical shear perimeter

{{

}}2(a),(c)

When openings are located at less than 10 times the slab thickness from a column, ACI 318 Section 11.11.6 requires that the critical perimeter be reduced, as shown by ACI 318 Figure R11.11.6.

When the concentrated load exceeds $\phi V_c (\phi = 0.75)$, shear reinforcement consisting of stirrups is provided. In this case, the nominal punching shear strength is calculated by Equation 8-10 above, where:

$$V_c = 2\sqrt{f_c'}b_o d \qquad \text{Equation 8-15}$$

$$V_s = \frac{A_v f_{yt} d}{2} \leq 4\sqrt{f_c'}b_o d \qquad \text{Equation 8-16}$$

where A_v is the cross-sectional area of the legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of the loaded area; i.e., along the critical perimeter, b_o.

When significant unbalanced moment is transferred between the slab and column, ACI 318 Section 11.11.7 requires that a portion of the unbalanced moment be transferred as shear in the critical perimeter. The procedure is explained in the commentary to ACI 318 Section 11.1.7.

8.11.3 Shear in T-Beams and Columns

The shear strength is calculated using the same equations for slabs and the following requirements:

- For beams, b_w is the width of the stem only.
- Whenever V_u exceeds $0.5\phi V_c$, shear reinforcement is provided with minimum area given by ACI 349 Equation 11-13 shown below.

$$V_{v,min} = 0.75\sqrt{f_c'}\frac{b_w s}{f_{yt}} \qquad \text{Equation 8-17}$$

- Shear reinforcement consists of closed stirrups with maximum spacing, s, limited to 0.5*d* or 24 in.
- For columns and compression reinforcement in beams, the size and spacing of the shear reinforcement also comply with the requirements for lateral ties for compression members in Section 8.11.5.

8.11.4 Torsion in T-Beams

The requirements for torsional members presented in ACI 349 Section 11.5 (same as ACI 318-08) apply to beams subject to torsional moment. The T-Beams are part of floor slabs and, hence, torsion in these T-beams may result from any unbalanced moment in the slab that they are part of. The magnitude of these potential torsional effects is considered to be small enough to not cause reduction in strength per ACI 349 Section 11.5 and, hence, the torsional effects on T-Beams is permitted to be negligible in the design methodology.

8.11.5 In-Plane Shear in Diaphragms and Basemats

In-plane shear in diaphragms is part of the seismic load path for transferring seismic forces to the vertical elements (i.e., walls). Therefore, the design is based on the seismic provisions for diaphragms in ACI 349 Section 21.11.

The nominal in-plane shear capacity of diaphragms is obtained using ACI 349 Equation 21-9, shown below:

$$V_n = A_{cv}(2\sqrt{f_c'} + \rho_t f_y) \le 8A_{cv}\sqrt{f_c'} \qquad \text{Equation 8-18}$$

Where A_{cv} (in.^{2}) is the gross area of the diaphragm cross section and ρ_t is the reinforcement ratio of the transverse rebar (parallel to the section cut). The shear strength is obtained by multiplying V_n by $\phi = 0.75$.

8.11.6 In-Plane Shear Transfer through Shear Friction

Shear friction is evaluated at construction joints where fresh concrete is placed against previously hardened concrete. In this case, a weak plane may exist at the joint or in concrete immediately adjacent to the joint. The construction process is reviewed to identify these locations.

Shear transfer between concrete slabs and SC walls is covered in Section 7.11.

From ACI 349 Section 11.6, the nominal shear-friction capacity along the critical section is calculated by:

$$V_n = (A_v f_y + N_u)\mu \qquad \text{Equation 8-19}$$

Where:

N_u (lb) is the axial load acting on the shear plane where shear, V_u, is transferred. N_u is positive for permanent compression loads and negative for tension loads. Compression loads resulting from seismic forces are not included since these loads are not permanent.

μ = 1.4 for concrete placed monolithically; 1.0 for concrete placed against hardened concrete with surface intentionally roughened as specified in ACI 349 Section 11.6.4.3; or, 0.6 otherwise.

A_v (in^2) is the total reinforcement crossing the critical section

A_c (in^2) is the cross section area of the critical section.

Equation 8-19 is a modified version of ACI 318 Equation (11-25) in which the effect of axial load, N_u, on shear-friction capacity is included. When N_u is permanent compression (positive), this force increases the shear-friction capacity; however, when it is tension (negative), it reduces the shear-friction capacity and thus, additional reinforcement is needed. This approach is consistent with ACI 318, Section 11.6.7.

For concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in ACI 349 Section 11.6.4.3:

$$V_n \leq \min[0.2f'_c A_c, (480 + 0.08f'_c)A_c, 1600A_c] \qquad \text{Equation 8-20}$$

For other cases:

$$V_n \leq \min[0.2f'_c A_c, 800A_c] \qquad \text{Equation 8-21}$$

The shear-friction strength is calculated by multiplying V_n by $\Phi = 0.75$.

8.12 Flexural Capacity

This applies to the out-of-plane bending of floor slabs and basemat, biaxial bending of columns, and major axis bending of T-beams. These members are subject to combined flexure and axial forces; therefore, the design procedure for beam or columns in ACI 349 Chapter 10 is used.

8.12.1 General Methodology

The combined flexural-axial load (P-M) strength is represented through P-M interaction diagrams for bending in one direction. The detailed procedure is shown in ACI 349 Appendix B. The calculations follow the basic design assumptions of ACI 349 Section 10.2 as summarized below.

- The strain in reinforcement and concrete is assumed to be directly proportional to the distance from the neutral axis. For simplicity, the strain distribution is defined by the extreme fiber longitudinal compressive strain ε_c and the extreme longitudinal reinforcing strain ε_t.
- The maximum usable strain at extreme concrete compression fiber is assumed equal to 0.003. The extreme fiber compressive strain is limited to 0.002 for out-of-plane flexure of slabs and diaphragms collectors.
- The stress-strain relationship in reinforcement is assumed to be elastic, perfectly plastic with a maximum stress equal to the yield stress, f_y. Strain hardening of the reinforcement is neglected. Conservatively, the maximum steel strain is limited to the minimum elongation required for A706 bars, which is 0.10 (Reference 10.1.25).

- The tensile strength of concrete is neglected in axial and flexural capacity calculations.

The Hognestad parabolic concrete stress strain model (Reference 10.1.24) is used to represent the concrete compressive strength. This model is defined by the following parameters:

$$\begin{aligned} f_c(\varepsilon) &= 0 && \text{for } 0 \le \varepsilon \\ &= -0.85 f_c' \left(2\frac{\varepsilon}{\varepsilon_o} - \left(\frac{\varepsilon}{\varepsilon_o}\right)^2 \right) && \text{for } \varepsilon_o < \varepsilon \le 0 \\ &= -0.85 f_c' \left(1 - 0.15 \frac{\varepsilon - \varepsilon_o}{-0.0038 - \varepsilon_o} \right) && \text{for } -0.003 < \varepsilon \le \varepsilon_o \end{aligned} \qquad \text{Equation 8-22}$$

where,

$$\varepsilon_o = -2\frac{f_c'}{E_c},$$

ε_o is the strain, positive in tension,

f_c' (psi) is the compressive strength, and

E_c (psi) is the concrete modulus defined as $E_c = 57000\sqrt{f_c'}$.

In accordance with ACI 349 Section 10.3, the balanced strain condition exists when the concrete compressive strain is 0.003 and the extreme rebar strain is at yield, with a strain of $\varepsilon_y = \frac{f_y}{E_s}$ with E_s = 29,000 ksi. Sections with a compressive strain of 0.003 and an extreme tensile strain at equilibrium equal or less than the yield strain of the reinforcement steel are considered to be compression-controlled. Sections with a compressive strain of 0.003 and an extreme tensile strain at equilibrium of 0.005 or more are considered to be tension-controlled. From ACI 349 Section 9.3, the strength reduction factor, ϕ, for compression members without spiral reinforcing is:

Tension-controlled	$\varepsilon_t \ge 0.005$	$\phi = 0.9$	Equation 8-23
Transition	$0.002 < \varepsilon_t < 0.005$	$\phi = 0.65 + 0.25\dfrac{\varepsilon_t - \varepsilon_y}{0.005 - \varepsilon_y}$	
Compression-controlled	$\varepsilon_t \le 0.002$	$\phi = 0.65$	

For members with an axial compression less than $0.10 f_c{}' A_g$, the section design is performed such that ε_t at the nominal strength is not less than 0.004.

Per ACI 349 Section 10.3.6, the compression strength for members with tie reinforcement conforming to ACI 349 Section 7.10.5 is limited to:

$$\varphi P_{n,max} = 0.8 \phi P_o \quad \text{Equation 8-24}$$

Where ϕP_o is the axial strength at zero eccentricity; that is, axial strength corresponding to a uniform strain in the section equal to the strain at peak compressive stress.

8.12.2 Out-of-Plane Flexural Capacity of Slabs and Basemat

The P-M interaction diagrams are built for one foot of section using the methodology described in Section 8.12.1.

When the axial loads include collector forces, the usable compression strain is limited to 0.002.

8.12.3 Flexural Capacity of T-Beams

The P-M interaction diagrams are built for the member cross-section. An effective width of the slab for the T-beams is included in the member cross-section for both compression and tension in the effective slab width.

The slab effective width requirements are included in ACI 318 Section 8.12.

Based on ACI 349 Section 10.6.6, the flexural reinforcement in the flange is distributed over a length equal to the minimum of the effective flange width and a width equal to one-tenth of the span.

8.12.4 Biaxial Bending of Columns

Columns subject to significant bending in one direction while subject to relatively small bending in the orthogonal direction are designed using P-M interaction diagrams for uniaxial bending (Section 8.12.1) and, when required, checked for strength in the orthogonal direction. Columns subject to equally significant bending moments in two orthogonal directions are designed for biaxial bending.

Biaxial bending interaction diagrams are developed using the methodology in Section 8.12.1 and varying the angle of the neutral axis. As an alternative, any simplified method available in the literature can be used to design the column for biaxial bending. The following methods are commonly used to design rectangular columns subjected to biaxial bending:

- Bresler reciprocal load method, described in Appendix D of ACI SP-17(14) (Reference 10.1.23).
- Load contour method, also described in Appendix D of ACI SP-17(14).
- The equivalent eccentricity method, described in Section 11-7 of MacGregor, (Reference 10.1.17).

Slenderness effects are evaluated for columns with clear height to thickness ratio higher than 7.3. For these columns, it is permitted to ignore slenderness effects if the following equation is satisfied (ACI 349 Equation 10-7):

$$\frac{kl_u}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right) \leq 40 \qquad \text{Equation 8-25}$$

where,

k is the effective length factor obtained as described below,

r is the radius of gyration, which can be taken as 0.3h, where h is the column dimension in the direction where stability is being considered,

M_1 is the smaller factored end moment, taken as positive if the member is bent in single curvature, and negative if bent in double curvature,

M_2 is the larger factored end moment, taken always positive, and

l_u is the unsupported column height.

The effective length factor k can be conservatively taken equal to 1.0. More accurate values can be obtained using the Jackson and Moreland Alignment Charts for non-sway frames shown in ACI 349 Figure R10.10.1.1.a.

If Equation 8-25 is not satisfied, the design of the member is performed using the factored forces and moments from any of the following procedures:

- Nonlinear second-order analysis, satisfying ACI 349 Section 10.10.3
- Elastic second-order analysis, satisfying ACI 349 Section 10.10.4
- Moment magnification procedure, satisfying ACI 349 Section 10.10.5.

If the moment magnification procedure is used, columns are designated as non-sway columns; thus, the moments are magnified using ACI 349 Section 10.10.6.

8.12.5 Collector Capacity

P-M interaction diagrams of collectors are calculated according to the procedure described in Section 8.12.1. The width of the collectors is the same as the wall thickness and the depth equal to the slab thickness. The calculated reinforcement is placed within this width. The usable compression strain is limited to 0.002 to avoid transverse reinforcement requirements per ACI 349 Section 21.11.7.5.

8.13 Demand to Capacity Ratio

Demand-to-capacity ratio (DCR) is calculated by dividing the total demand by the capacity as shown by Equation 8-26. Note that DCR is less than one for acceptable design.

$$DCR = \frac{D}{C} \qquad \text{Equation 8-26}$$

where,

D = total demand, for specific actions, acting on a member specific location (e.g., at a panel section or at a section cut), and

C = capacity of a member specific location (e.g., at a panel section or at a section cut).

The axial-bending (P-M) interaction capacity is calculated by scaling the total P-M demand to the capacity curve, as shown in Figure 8-24.

Figure 8-24 Demand to capacity ratio for P-M demand

{{

}}2(a),(c)

8.14 Total Reinforcement

An iterative design process is implemented in which the reinforcement is incremented until the member capacities exceed the demand. In P-M design checks, the demand corresponds to the enveloped demand obtained as described in Section 8.10.

8.14.1 Slabs and Diaphragms

Slabs are subject to in-plane shear, out-of-plane moment, out-of-plane shear, and axial load. The total slab reinforcement in each direction is the resultant of the reinforcement required for each of the demand types. To avoid excessive conservatism, the reinforcement due to the different demand types is added at the same slab region (i.e., reinforcement is not varied at different locations in the slab).

Since the axial load is considered together with the out-of-plane moment and out-of-plane shear (i.e., P-M and P-V interactions), the horizontal reinforcement is calculated due to P-M interaction and in-plane shear. The vertical shear reinforcement (through the thickness), is calculated due to P-V interaction.

Considering the slab panel shown in Figure 8-25, subject to combined in-plane shear, out-of-plane moment (P-M) and out-of-plane shear (P-V), the required reinforcement ratio (both faces) crossing the XZ plane (Y-reinforcement) and the YZ plane (X-reinforcement), is shown in Table 8-6.

The reinforcement used in the P-M interaction diagrams is chosen so that the DCR obtained as explained in Section 8.13 is close to one.

The total reinforcement ratio for the slab panel shown in Figure 8-25, calculated as described above, is shown at the bottom of Table 8-6.

Out-of-plane shear reinforcement for slabs conforms with reinforcement requirements described in Section 8.10.4.

Table 8-6 Reinforcement design in slabs

Member section	**Demands from load**	**Reinforcement**		
		X (longitudinal)	**Y (transverse)**	**Z (thickness)**
XZ plane Longitudinal	In plane shear	$\rho_{X,IP,XZ}$		
	Out of plane shear			$\rho_{Z,OP,XZ}$
	Out of plane moment		$\rho_{Y,OP,XZ}$	
YZ plane Transverse	In plane shear		$\rho_{Y,IP,YZ}$	
	Out of plane shear			$\rho_{Z,OP,YZ}$
	Out of plane moment	$\rho_{\mathrm{X,OP,YZ}}$		
Total rebar calculation		$\rho_{x,IP,XZ} + \rho_{X,OP,YZ}$	$\rho_{Y,OP,XZ} + \rho_{Y,IP,YZ}$	$\max(\rho_{Y,OP,XZ}, \rho_{Z,OP,YZ})$

Figure 8-25 Force and moment demand at two perpendicular section cuts in a slab

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}}2(a),(c)

8.14.2 T-Beams and Columns

Similar to the slabs, the longitudinal reinforcement is calculated based on P-M interaction diagrams. The shear reinforcement is calculated due to P-V interaction.

The selected flexural reinforcements also satisfy the minimum reinforcement requirement of Section 8.10.2.2 for T-beams and 8.10.2.3 for columns.

Also, as shown in Section 8.11.3, a minimum shear reinforcement is required where V_u exceeds $0.5\phi V_c$.

8.14.3 Basemats

As described in Section 8.3.1, the basemat is subject to out-of-plane forces and moments. The longitudinal and transverse reinforcement for out-of-plane flexure is obtained in the same way as for the slabs in Section 8.14.1. The out-of-plane shear reinforcement, if any, is also obtained as shown in Section 8.14.1.

Shear reinforcement extends as close as possible to tension and compression faces of the basemat and hook around longitudinal reinforcement (Reference 10.1.19). A typical configuration is a single-leg stirrup having 135-degrees hook on one end and 90-degrees hook on the opposite end.

Although not required by ACI 349, it is considered to include vertical closure reinforcement and skin reinforcement along any free edges of the basemat (Reference 10.1.19).

9.0 Conclusions

This topical report proposes an updated evaluation methodology implementing recent developments in the design and evaluation of complex safety-related Seismic Category I and II structures for applicability to the new generation of SMR designs. It implements the soil library methodology for evaluation of complex structures, as presented in the recently approved NuScale Topical Report TR-0118-58005, “Improvements in Frequency Domain Soil-Structure-Fluid Interaction Analysis” (Reference 10.1.26). It further describes the development of analytical models with damping values and stiffness properties based on the actual stress state of the members under seismic load; and describes the member design process for load combinations that involve seismic loads. It implements code-specified effective stiffness values to the ANSYS finite element models.

The methodology presented in this topical report has also incorporated the use of SC walls that can be connected to each other and anchored to a traditionally-constructed RC basemat. The methodology is based on the requirements of the American Institute of Steel Construction, ANSI/AISC N690-18 and the Specification for Structural Steel Buildings, ANSI/AISC 360-16 as required by ANSI/AISC N690-18. The presented methodology applies to straight SC walls with steel-headed stud anchors and structures using load and resistance factor design.

The methodology includes requirements and rules for designing and detailing SC walls, including response and sizing requirements under impactive and impulsive loading. It also provides general guidance for the SC walls connections design. Some of the guidance presented herein is only applicable to certain SC wall connections configurations. Thus, a user is required to employ judgment to choose applicable failure limits to the design configuration case.

The design methodology is applicable to a typical SMR Reactor Building, CRB, and RWB. The design methodology is based on Appendix N9 of AISC N690, including its commentary. The methodology for SC wall connection design is provided in Section 7.0.

Finally, this topical report supplements the analysis and design methodology for RC members such as basemats, slabs, and roof members that interact with SC walls.

10.0 References

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Enclosure 3:

Affidavit of Mark W. Shaver, AF-107606

NuScale Power, LLC
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www.nuscalepower.com

NuScale Power, LLC

AFFIDAVIT of Mark W. Shaver

I, Maek W. Shaver, state as follows:

(1) I am the Licensing Manager of NuScale Power, LLC (NuScale), and as such, I have been specifically delegated the function of reviewing the information described in this Affidavit that NuScale seeks to have withheld from public disclosure, and am authorized to apply for its withholding on behalf of NuScale

(2) I am knowledgeable of the criteria and procedures used by NuScale in designating information as a trade secret, privileged, or as confidential commercial or financial information. This request to withhold information from public disclosure is driven by one or more of the following:

(a) The information requested to be withheld reveals distinguishing aspects of a process (or component, structure, tool, method, etc.) whose use by NuScale competitors, without a license from NuScale, would constitute a competitive economic disadvantage to NuScale.

(b) The information requested to be withheld consists of supporting data, including test data, relative to a process (or component, structure, tool, method, etc.), and the application of the data secures a competitive economic advantage, as described more fully in paragraph 3 of this Affidavit.

(c) Use by a competitor of the information requested to be withheld would reduce the competitor's expenditure of resources, or improve its competitive position, in the design, manufacture, shipment, installation, assurance of quality, or licensing of a similar product.

(d) The information requested to be withheld reveals cost or price information, production capabilities, budget levels, or commercial strategies of NuScale.

(e) The information requested to be withheld consists of patentable ideas.

(3) Public disclosure of the information sought to be withheld is likely to cause substantial harm to NuScale's competitive position and foreclose or reduce the availability of profit-making opportunities. The accompanying topical report reveals distinguishing aspects about the methodology by which NuScale develops its building design and analysis for safety-related structures.

NuScale has performed significant research and evaluation to develop a basis for this methodology and has invested significant resources, including the expenditure of a considerable sum of money.

The precise financial value of the information is difficult to quantify, but it is a key element of the design basis for a NuScale plant and, therefore, has substantial value to NuScale.

If the information were disclosed to the public, NuScale's competitors would have access to the information without purchasing the right to use it or having been required to undertake a similar expenditure of resources. Such disclosure would constitute a misappropriation of NuScale's intellectual property, and would deprive NuScale of the opportunity to exercise its competitive advantage to seek an adequate return on its investment.

(4) The information sought to be withheld is in the enclosed topical report entitled “Building Design and Analysis Methodology for Safety-Related Structures.” The enclosure contains the designation “Proprietary" at the top of each page containing proprietary information. The information considered by NuScale to be proprietary is identified within double braces, "{{ }}" in the document.

(5) The basis for proposing that the information be withheld is that NuScale treats the information as a trade secret, privileged, or as confidential commercial or financial information. NuScale relies upon

the exemption from disclosure set forth in the Freedom of Information Act ("FOIA"), 5 USC § 552(b)(4), as well as exemptions applicable to the NRC under 10 CFR §§ 2.390(a)(4) and 9.17(a)(4).

(6) Pursuant to the provisions set forth in 10 CFR § 2.390(b)(4), the following is provided for consideration by the Commission in determining whether the information sought to be withheld from public disclosure should be withheld:

(a) The information sought to be withheld is owned and has been held in confidence by NuScale.

(b) The information is of a sort customarily held in confidence by NuScale and, to the best of my knowledge and belief, consistently has been held in confidence by NuScale. The procedure for approval of external release of such information typically requires review by the staff manager, project manager, chief technology officer or other equivalent authority, or the manager of the cognizant marketing function (or his delegate), for technical content, competitive effect, and determination of the accuracy of the proprietary designation. Disclosures outside NuScale are limited to regulatory bodies, customers and potential customers and their agents, suppliers, licensees, and others with a legitimate need for the information, and then only in accordance with appropriate regulatory provisions or contractual agreements to maintain confidentiality.

(c) The information is being transmitted to and received by the NRC in confidence.

(d) No public disclosure of the information has been made, and it is not available in public sources. All disclosures to third parties, including any required transmittals to NRC, have been made, or must be made, pursuant to regulatory provisions or contractual agreements that provide for maintenance of the information in confidence.

(e) Public disclosure of the information is likely to cause substantial harm to the competitive position of NuScale, taking into account the value of the information to NuScale, the amount of effort and money expended by NuScale in developing the information, and the difficulty others would have in acquiring or duplicating the information. The information sought to be withheld is part of NuScale's technology that provides NuScale with a competitive advantage over other firms in the industry. NuScale has invested significant human and financial capital in developing this technology and NuScale believes it would be difficult for others to duplicate the technology without access to the information sought to be withheld.

I declare under penalty of perjury that the foregoing is true and correct. Executed on October 6, 2021.

Mark W. Shaver

Mark W. Shaver