RAIO-104576

July 16, 2021

Docket: 99902078

U.S. Nuclear Regulatory Commission
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SUBJECT: NuScale Power, LLC Response to NRC Request for Additional Information (RAI No. 9833 and 9834) on the NuScale Topical Report, "Building Design and Analysis Methodology for Safety-Related Structures," TR-0920-71621, Revision 0

REFERENCES:
1. NRC Letter Final Request for Additional Information No. 0002 (eRAI Nos. 9833 and 9844), dated May 10, 2021, RAI# 9833 and 9834
2. NuScale Topical Report "Building Design and Analysis Methodology for Safety-Related Structures," TR-0920-71621, Revision 0 - ML20353A406, dated December 2020

The purpose of this letter is to provide NuScale's response to NRC Requests for Additional Information (RAI), RAI# 9833 and 9834, noted in the References above. The responses to the individual RAI questions are provided in the attached Enclosures.

This letter contains NuScale's response to the following RAI Questions from NRC RAI# 9833 and 9834:

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Enclosures are grouped with all propietary version responses first, followed by all nonproprietary version responses. NuScale requests that the proprietary versions be withheld from public disclosure in accordance with the requirements of 10 CFR §2.390. The enclosed affidavit supports this request.

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

Please contact Liz English at 541-452-7333 or at eenglish@nuscalepower.com if you have any questions.

Sincerely,

Mark W. Shaver

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Licensing Manager
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Enclosure 1: NuScale Response to NRC Request for Additional Information RAI# 9833 and 9834, proprietary

Enclosure 2: NuScale Response to NRC Request for Additional Information RAI# 9833 and 9834, nonproprietary

Enclosure 3: Affidavit of Mark Shaver, AF-104578

Enclosure 1:

NuScale Response to NRC Request for Additional Information RAI# 9833 and 9834, proprietary

Enclosure 2:

NuScale Response to NRC Request for Additional Information RAI# 9833 and 9834, nonproprietary

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9834
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-1

Requirement

10 CFR Part 50, Appendix A, General Design Criteria 2, as it relates to the design of seismic Category I structures, systems and components (SSCs).

10 CFR Part 50, Appendix S, provides criteria for the implementation of GDC 2 with respect to earthquakes and requires, in part, that the safety functions of SSCs important to safety must be assured during and after the vibratory ground motion associated with the Safe Shutdown Earthquake Ground Motion, and that the evaluation must take into account soil-structure interaction (SSI) effects.

DSRS Section 3.7.2 states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in SSI analyses.

Issue

In Section 4.0 of the TR, the applicant assumes the soil surrounding the triple building (TRB) model TRB-Static is of soil type 11 (soft soil) with stiffness reduced by 50% to account for the settlement effects. However, no basis for the assumption is given. It is not clear why 50% reduction of the stiffness of the surrounding soil mass would appropriately account for both immediate settlement and long-term settlement due to load exerted by the TRB.

Request

Please provide the basis for assuming that a 50% reduction of the stiffness of the surrounding soil mass would be adequate to bound the predicted effects from both immediate and long-term settlements.

NuScale Response:

The basis for the 50 percent soil stiffness reduction is provided in DCA Section 3.8.5.5.5 (ML20225A154). The soil stiffnesses are reduced by 50 percent to amplify the effect of differential movements or settlements. The 50 percent reduction in soil stiffness includes the areas below the triple building basemats and is extended to the entire free-field soil model.

In accordance with Design Specific Review Standard (DSRS) 3.7.2, deterministic differential settlement analysis is performed for the lower bound (soft-stiffness) generic soil profile for which the shear modulus is reduced using the following equation:

$$G_{LB} = G_{BE} / (1 + COV)$$

Where G_{LB} and G_{BE} are the shear moduli for the lower bound (LB) and best estimate (BE) soil profiles and COV is the coefficient of variation considered appropriate for the site materials.

For well-investigated sites, the DSRS states that COV should be no less than 0.5 and for not well-investigated sites, it should be at least 1.0. DSRS 3.7.2 further states that in no case should the lower bound shear modulus cause greater foundation settlement (based on standard foundation analysis) than that under static loads.

For differential settlement analysis, the generic soil profile 11 (soft soil) is used and the stiffness is further reduced by 50 percent, which is equivalent to taking COV as 1.0. Given the employment of the softest soil profile, it has been deemed conservative enough to use 1.0 as the COV value to bound all settlement induced effects.

Impact on Topical Report:

There are no impacts to Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, as a result of this response.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9834
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-2

Requirements

10 CFR Part 50, Appendix A, General Design Criteria 2, as it relates to the design of seismic Category I structures, systems and components (SSCs).

10 CFR Part 50, Appendix S, provides criteria for the implementation of GDC 2 with respect to earthquakes and requires, in part, that the safety functions of SSCs important to safety must be assured during and after the vibratory ground motion associated with the Safe Shutdown Earthquake Ground Motion, and that the evaluation must take into account soil-structure interaction (SSI) effects.

DSRS Section 3.7.2 states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in SSI analyses.

Issues

a) Three different surrounding media have been described in the TR representing soil, rock, and hard soil (soil types 11, 9, and 7, respectively). It is not clear what are the differences among these three different media types.

b) It has been assumed that the surrounding medium can be represented by only one material type; either a soil type 9 (hard rock), a soil type 7 (rock), or a soil type 11 (soft soil). It is not clear in the TR whether a layered medium (with several soil or rock layers, or both; such as soil underlain by rock) can be considered as the surrounding medium.

c) It is not clear whether the soil library models account for the nonlinear behavior at the structure-surrounding medium interfaces and also in the surrounding medium, especially for soil type 9 (hard rock). At a scale of the reactor, the hard rockmass may have naturally occurring

discontinuities, such as, rock joints, bedding planes, or faults. These discontinuities can make the surrounding medium a discontinuum with potential for sliding and opening along and/or across the discontinuities exhibiting nonlinear behavior. Similarly, structure-medium interfaces may slide exhibiting nonlinear behavior. It is not clear how the potential discontinuum medium with associated nonlinear response would be addressed with the strategy described in Section 4 of the TR to develop the in-structure response spectra and design methodology for Seismic Category I and II structures, systems, and components.

Request

a) Describe the characteristics of each soil type, especially those characteristics that separate among three soil types.

b) Clarify whether a layered medium with either several soil layers, rock layers, or soil layers underlain by rock layers can be analyzed with the strategy described in the TR.

c) Discuss in detail the strategy to develop the in-structure response spectra and design methodology, especially in hard rock (soil type 9), where the surrounding medium exhibits nonlinear and discontinuous behavior.

NuScale Response:

a) Characteristics of the generic soil profiles are described in DCA Section 3.7.1.3.1 (ML20225A154). The soil profiles are updated to be strain compatible for Certified Seismic Design Response Spectra (CSDRS) and CSDRS for High Frequency (CSDRS-HF) seismic loads as described in DCA Section 3.7.1.3.2. In-depth variations of S-wave velocity, density, damping and the existence of ground water are the major differences between the three strain compatible soil profiles.

b) The three strain compatible generic soil profiles consist of multiple layers with varying parameters in depth. Specifically, for Soil-11 the soft soil layers are underlain by bedrock where the impedance jump is significantly large. Thus, the outlined strategy in TR-0920-71621 is valid for any layered medium.

c) Soil library methodology, presented in TR-0118-58005 (ML20353A439), is based on the assumption that the soil surrounding the excavation volume is continuous without any cavities or faults and made of linear elastic material. To address the soil material model non-linearity, the generic soil profiles are updated to be strain compatible as described in DCA Section 3.7.1.3.2.

Soil-structure interface discontinuity is to be addressed through a combination of linear analysis and if needed nonlinear transient analyses. Transient stability analyses are described in DCA 3.8.5 (ML20225A154). The necessity for nonlinear transient analysis is decided based on a factor-of-safety to be determined from the ratio of the driving force to the resisting force in the linear analysis.

Discontinuity within the soil media surrounding the excavation volume can be identified through an investigation of site-specific geologic features (site characterization). This case-dependent condition can be addressed by explicitly modeling the discontinuity and its vicinity, together with the power plant structures and following the same approach used for stability analysis as described above.

Furthermore, DCA COL Items 3.7-3, 3.7-9, 3.7-11, 3.7-13, and 3.8-3 address site-specific conditions and nonlinear effects that are not bound by the parameters that formed the design basis.

Impact on Topical Report:

There are no impacts to Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, as a result of this response.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9834
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-3

Requirement

10 CFR Part 50, Appendix A, General Design Criteria 2, as it relates to the design of seismic Category I structures, systems and components (SSCs).

10 CFR Part 50, Appendix S, provides criteria for the implementation of GDC 2 with respect to earthquakes and requires, in part, that the safety functions of SSCs important to safety must be assured during and after the vibratory ground motion associated with the Safe Shutdown Earthquake Ground Motion, and that the evaluation must take into account soil-structure interaction (SSI) effects.

DSRS Section 3.7.2 states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in SSI analyses.

Issue

In Section 4.1, "Determination of Effective Stiffness and Damping," of the TR, a damping value of 4% was used, as shown in Table 4-1, for steel-plate composite (SC) walls at Response Level 1. As stated in the TR, this value is based on the report "S. Epackachi and A. S. Whittaker, "Experimental, numerical, and analytical studies on the seismic response of steel-plate concrete (SC) composite walls," Multidisciplinary Center for Earthquake Engineering Research (MCEER) 2016, as the specific value to be used in the analysis is not given in ASCE 4-16. However, Table 3-1, "Specified Damping Values for Dynamic Analysis," of ASCE 43-19, gives a damping value of 3% of the critical damping for the Response Level 1 for the SC walls. The TR referred to ASCE 43-05, not ASCE 43-19, which is the latest version.

Request

The staff requests the applicant to provide the rationale as to why a value of 4% of the critical damping would be appropriate for SC walls, given the latest revision of ASCE 43-19 gives a value of 3%.

NuScale Response:

ASCE 43-19 was not yet published during the development of the TR-0920-71621. TR-0920-71621 is revised to reflect latest ASCE 43-19 and the use of 3 percent of the critical damping for Response Level 1 instead of 4 percent for steel-plate composite walls.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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~~43-05~~ (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~~43-05~~, 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 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 4 percent is used in this methodology based on Reference 10.1.30.~~ The damping ratios for SC walls are provided in Table 4-1.

~~Use of SC walls in structural systems has been studied by many industry researchers. Among them is a study performed by Multidisciplinary Center for Earthquake Engineering Research (MCEER) in 2016 (Reference 10.1.30). This research included both experimental and numerical studies of the seismic response of the SC walls.~~

~~The equivalent viscous damping for each wall is calculated using the strain energy and the dissipated energy method cycle (the enclosed area of the force-displacement curve) as noted in Equation 5-1 of Reference 10.1.30.~~

~~The results of the equivalent viscous damping for tested walls is shown in Figure 5-10 of Reference 10.1.30. The damping ratio for SC walls ranges from 4 percent to more than 15 percent (Reference 10.1.30). For the uncracked concrete condition 4 percent damping ratio is used. The 4 percent represents the lowest damping ratio of the first data points in the pre-peak strength region.~~

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~~4~~	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~~43-05~~ 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~~9~~, a representative in-column input

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~~43-05~~ (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:

10.0 References

10.1.1 American Society of Civil Engineers, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities," ASCE 43-19~~05~~, Reston, VA, ~~2005~~2019.

10.1.2 American Society of Civil Engineers, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," ASCE 4-16, Reston, VA, 2016.

10.1.3 American National Standards Institute/American Institute of Steel Construction, "Specification for Safety-Related Steel Structures for Nuclear Facilities," ANSI/AISC N690-18, Chicago, IL, 2018.

10.1.4 American Concrete Institute, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," ACI 349-13, Farmington Hills, MI, 2014.

10.1.5 American Institute of Steel Construction, "Design of Modular Steel-Plate Composite Walls for Safety-Related Nuclear Facilities," Steel Design Guide 32, Chicago, IL, 2017.

10.1.6 U.S. Nuclear Regulatory Commission, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," Regulatory Guide 1.142, Rev. 3, June 2020.

10.1.7 U.S. Nuclear Regulatory Commission, "Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components," Regulatory Guide 1.122, Rev. 1, February 1978.

10.1.8 ANSYS, Inc. Mechanical APDL Element Reference, Release 18.2.

10.1.9 American Concrete Institute, "Building Code Requirements for Structural Concrete and Commentary," ACI 318-14, Farmington Hills, MI, 2014.

10.1.10 American Concrete Institute, "Building code Requirements for Structural Concrete," ACI 318-05, Farmington Hills, MI, August 2005.

10.1.11 Department of Energy, "Accident Analysis for Aircraft Crash Into Hazard Facilities," DOE-STD-3014-2006 (Reaffirmed), May 29, 2006.

10.1.12 Nuclear Energy Institute, "Methodology for Performing Aircraft Impact Assessment for New Plant Design," NEI 07-13, Revision 8P, April 2011.

10.1.13 Bruhl, J.C., A.H. Varma, and W.H. Johnson, "Design of Composite SC Walls to Prevent Perforation from Missile Impact," International Journal of Impact Engineering, Vol 75, pp 75-87, January 2015.

10.1.14 Biggs, J.M., *Introduction to Structural Dynamics*, McGraw-Hill, Inc., New York, NY, 1964.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9834
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-4

Requirement

10 CFR Part 50, Appendix A, General Design Criteria 2, as it relates to the design of seismic Category I structures, systems and components (SSCs).

10 CFR Part 50, Appendix S, provides criteria for the implementation of GDC 2 with respect to earthquakes and requires, in part, that the safety functions of SSCs important to safety must be assured during and after the vibratory ground motion associated with the Safe Shutdown Earthquake Ground Motion, and that the evaluation must take into account soil-structure interaction (SSI) effects.

DSRS Section 3.7.2 states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in SSI analyses.

Issue

The text in many places in the TR may be misleading or incorrect. For example:

1) In Section 5.5.2.1, "Method 1 for Implementing Effective Stiffness Using SOLSH190 Elements," it is confusing which numbers are used to specify the outer layers: are they 1 and 2 or 1 and 3? Figure 5-2 shows they should be 1 and 3 and so does the equation on page 31. However, it seems the equations on pages 32 and 33 indicate that they are 1 and 2.

2) In Section 5.5.2, "Implementation of Effective Stiffness Values for Solid-Shell Elements," the text refers to AISC 4-16 which is not in the reference list.

3) On page 33 of the TR, the term "t1-lay1" has been defined twice, the second definition on line 3 appears to be incorrect.

4) On page 35, reference to Equation (5-12) and Equation (5-13) seems incorrect. Similarly, reference to Equation (5-17) seems incorrect on page 36.

5) Equation (5-29) relating the strain and stress vectors for an orthotropic material appears incorrect.

6) On page 25 of the TR for the condition "if Srxy > 2Scr," the reference to AISC N60-18 Equation A-N9-12 to calculate the effective shear stiffness appears incorrect

Request

1) Clarify the layer numbers used to refer the outer layers.

2) Correct the reference standard.

3) Clarify and, if necessary, correct the equations.

4) Verify that referenced equations (5-12), (5-13), and (5-17) are correct; if not, correct.

5) Clarify whether equation (5-29) for the orthotropic material is correct as written. If not, correct.

6) Clarify whether the referred equation in AISC 690-18 is correct. If not, correct.

NuScale Response:

Response to Request 1

TR-0920-71621 Figure 5-2, not revised as part of this response, shows the three-layered solid-shell element outer material layers as 1 and 3. This numbering applies to walls and slabs modeled with a single row of elements through the thickness.TR-0920-71621 is revised to state that the thickness of the outer layers (layers 1 and 3 in Figure 5-2) is calculated as: $t_{1,3} = (t - t_m)/2$.

TR-0920-71621 refers to walls and slabs modeled with two rows of solid-shell elements. In this context, as shown 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 “t1-lay1”, “t1-lay2”, “t2-lay1”and “t2-lay2”. TR-0920-71621 is revised for clarification .

Response to Request 2

ASCE 4-16 is listed as Reference 10.1.2 American Society of Civil Engineers, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," ASCE 4-16, Reston, VA, 2016. TR-0920-71621 is revised to include a cross-reference to Reference 10.1.2.

Response to Request 3

TR-0920-72621 is revised accordingly to state "t2-lay1" on line 3 instead of "t1-lay1".

Response to Request 4

TR-0920-72621 revised to reflect correct Equation references.

Response to Request 5

TR-0920-72621 Equation (5-29) is typed incorrectly. The flexibility or compliance matrix on the right side of Equation (5-29) should be as given by Equation (2-4) of the ANSYS Mechanical APDL Theory Reference.

Equation 5-29 is revised and new reference: ANSYS, Inc. *Mechanical APDL Theory Reference,* Release 18.2, added to TR-0920-71621.

Response to Request 6

TR-0920-71621 is revised to clarify the reference to AISC N690-18 Equation A-N9-14 if effective shear stiffness corresponds to the cracked shear stiffness.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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

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~~2~~.

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

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

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 ~~AISC~~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 ~~AISC~~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, $E \leq 10$ 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.

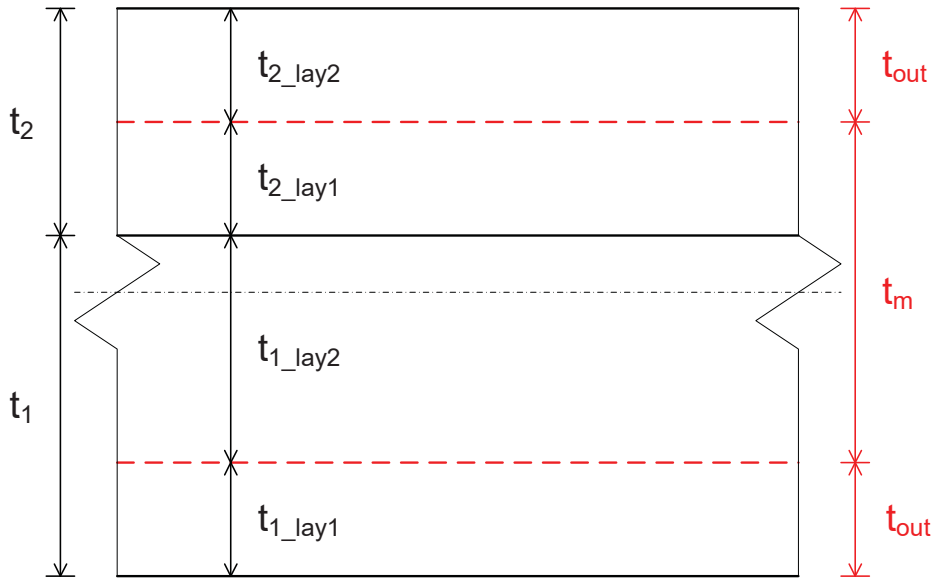
The thickness of the outer layers (layers 1 and 3~~2~~ in Figure 5-2) is calculated as

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

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 5-3 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:

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

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}	$E_sA_s + E_cA_c$	EA_{eff}	$EA_{eff} = E_sA_s + \frac{E_cA_c}{2}$ 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:

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

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

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 5-12~~Equation 5-11 and ~~Equation 5-13~~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 5-12~~Equation 5-11 and ~~Equation 5-13~~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:

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

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

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~~Equation 5-17~~ 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

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

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

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 10.1.31):

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

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.

10.1.28 National Institute of Standards and Technology, “Seismic design of cast-in-place concrete diaphragms, chords, and collectors: A guide for practicing engineers,” Second Edition, GCR 16-917-42, NEHRP Seismic Design Technical Brief No. 3, produced by the Applied Technology Council for the National Institute of Standards and Technology, Gaithersburg, MD, October 2016.

10.1.29 Klemencic, Ron, McFalane, Ian S., Hawkins, Neil M., and Nikolaou, Sissy, “Seismic design of reinforced concrete mat foundations: A guide for practicing engineers,” NEHRP Seismic Design Technical Brief No. 7, produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 12-917-22, August 2012.

10.1.30 Siamak Epackachi and Andrew S. Whittaker. Experimental, numerical, and analytical studies on the seismic response of steel-plate concrete (SC) composite walls, Multidisciplinary Center for Earthquake Engineering Research (MCEER), 2016.

10.1.31 ANSYS, Inc. Mechanical APDL Theory Reference, Release 18.2, August 2017.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9834
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-5

Requirement

10 CFR Part 50, Appendix A, General Design Criteria (GDC) 2, as it relates to the design of seismic Category I structures, systems and components (SSCs).

10 CFR Part 50, Appendix S, provides criteria for the implementation of GDC 2 with respect to earthquakes and requires, in part, that the safety functions of SSCs important to safety must be assured during and after the vibratory ground motion associated with the Safe Shutdown Earthquake Ground Motion, and that the evaluation must take into account soil-structure interaction (SSI) effects.

Standard Review Plan (SRP) NUREG−0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue

In the TR, the applicant used six strong motion earthquakes identified by the station name where it was recorded; namely, Capitola (CAP), Chi-Chi (CHI), El Centro (ELC), Izmit (IZM), Lucerne (LCN), and Yermo (YER). The seismic motion of each of the recorded earthquake was converted to the Certified Seismic Design Response Spectra (CSDRS) or Certified Seismic Design Response Spectra−high frequency (CSDRS−HF), and used selectively in the analysis. For example, only {{ }}2(a),(c) was used to calculate the in-plane shear and bending stresses and compared with the cracking limits given in Section 4.1, as shown in Figure 4-1. Similarly, {{ }}2(a),(c) was used as the only high frequency input motion (CSDRS−HF), as shown in Figure 4-1 of the TR. In Section 4.1.2, Effective Stiffness and Damping Ratio Assignment, it has been stated that “For soil types 7 and 9, a representative in-column input motion is considered as the SSE in load combination 4-1 (e.g. Capitola). For soil type 11, the

only in-column input motion is Lucerne." It is not clear what characteristics of each of these six CSDRSs would make one suitable for a specific analysis with a specific soil type, as described in the TR.

Request

The staff requests the applicant to provide the characteristics of each seismic motion record. Additionally, discuss the rationale and the unique characteristic(s) of each recorded seismic motion that distinguishes it suitable for a specific analysis with a specific soil type; e.g. why {{ }}2(a),(c) would be suitable for as CSDRS−HF but not others, why only {{ }}2(a),(c) can be used to determine whether a structural member develops cracks or not, and why {{ }}2(a),(c) (CSDRS−HF) can be used with soil type 11 (soft soil).

NuScale Response:

Characteristics of the six seismic motions are provided in DCA Section 3.7.1.1.2.1 (ML20225A154) as follows:

Yermo station: This first set of time histories was recorded at the Yermo Fire Station during the 1992 Landers Earthquake, which occurred on June 28, 1992 at 04:57 am (11:57 coordinated universal time [UTC]), with an epicenter near the town of Landers, California. It was a magnitude 7.3 moment magnitude scale (MMS) earthquake. The time step is 0.02 seconds and the duration is 43.98 seconds and the maximum PGA recorded is 0.245g.

Capitola station: The second set of time histories selected for spectrum compatible modification was recorded at station 47125 Capitola during the 1989 Loma Prieta Earthquake striking the San Francisco Bay Area of California on October 17, 1989 at 5:04 pm (October 18, 1989 at 00:04 UTC). It was a magnitude 6.9 MMS earthquake. The time step size of the recording is 0.005 seconds and the duration is 39.95 seconds. The maximum PGA recorded is 0.541g.

Chi-Chi station: The third set of time histories selected for spectrum compatible modification was recorded at Station TCU076 during the 1999 Chi-Chi Earthquake also known as the 921 Earthquake since it occurred on September 21, 1999 at 1:47 am (September 20, 1999 at 17:47 UTC) in central Taiwan. The time step size of the recording is 0.005 seconds and the duration is 89.995 seconds.It was a 7.6 MMS earthquake. The maximum PGA recorded is 0.416g.

Izmit station: The fourth set of time histories selected for spectrum compatible modification was recorded at Station Izmit during the 1999 Kocaeli Earthquake which struck northwestern Turkey

on August 17, 1999 at 3:02 am (00:02 UTC). It was a magnitude 7.4 MMS earthquake. The time step size of the recording is 0.005 seconds and the duration is 29.995 seconds. The maximum PGA recorded is 0.22g.

El Centro station: The fifth set of time histories selected for spectrum compatible modification was recorded at station 117 El Centro Array #9 during the 1940 Imperial Valley Earthquake which occurred on May 18, 1940 at 8:37 pm (May 19, 1940 at 05:35 UTC) in the Imperial Valley in southeastern Southern California. It was a magnitude 6.9 MMS earthquake. The time step size of the recording is 0.01 seconds and the duration is 39.99 seconds. The maximum peak ground acceleration recorded is 0.313g.

Lucerne station: This set of time histories was selected from a rock site for the generation of the seismic acceleration time histories to be compatible with the Certified Seismic Design Response Spectra-High Frequency (CSDRS-HF). These time histories were recorded at the Lucerne station during the 1992 Landers Earthquake which occurred on June 28, 1992 at 04:57 am (11:57 UTC), with an epicenter near the town of Landers, California. The 1992 Landers earthquake was a magnitude 7.3 MMS earthquake. The duration is 48.12 seconds and the time-step size is 0.005 seconds. The maximum PGA recorded is 0.818g.

In order to minimize modifications to a time history and to make its spectrum compatible, a recording was chosen with a frequency content as close as possible to the target spectrum. Also, the strong motion duration should be at least six seconds, which is one of the time history acceptance criteria.

Why would Lucerne be suitable for CSDRS−HF but not others?

The Lucerne recording has a higher frequency content (in the range of 20-50 Hz), thus making it the best candidate for matching the CSDRS-HF spectrum with minimal modification to the time series.

Why can only Capitola be used to determine whether a structural member develops cracks or not?

Cracking is evaluated based on the evaluation of stresses along multiple critical sections of the structural members of interest. All finite elements within a structural member are deemed as cracked or uncracked upon this evaluation. This type of assessment of the cracking along multiple critical sections per structural member suppresses the variations between different input motion characteristics. A parametric study, performed to assess the level of cracking within reactor building model using all CSDRS compatible input motions and Soil-7 and -11

libraries produced similar results and confirmed that all input motions lead to the cracking of the same structural members with the same soil type.Therefore, in order to evaluate cracking, any of the five input motions (already modified to be CSDRS compatible) can be used. Capitola was chosen because it was extensively used in the past (DCA) for subset analysis (e.g., soil-separation, sloshing, NPM analysis) as the controlling earthquake.

Why can Lucerne (CSDRS−HF) be used with soil type 11 (soft soil)?

For soil type 9, the only in-column input motion is Lucerne. Section 4.1.2 of TR-0920-71621 is revised to reflect this change. The Lucerne (CSDRS-HF) is selected based on the site properties. The CSDRS-HF represents a design response spectrum for hard rock sites typical of the Central Eastern United States (CEUS). The CSDRS-HF is used with soil type 9 because soil type 9 has the properties of a hard rock site, with a shear wave velocity of 8000 feet per second.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

(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~~9~~, a representative in-column input

motion is considered as the SSE in load combination 4-1 (e.g., Capitola). For soil type 9~~11~~, 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.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-6

Requirement

10 CFR Part 50, Appendix A, General Design Criteria (GDC) 1, 2 and 4 as it relates to the design of seismic Category I structures, systems and components (SSCs).

DSRS 3.8.4, "Other Seismic Category I Structures," states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 6.7.2, "Evaluation of the Local Response of Steel-Plate Composite Walls to Impactive and Impulsive Loads," of the TR, the applicant stated "*70 percent of the thickness for a RC wall was determined using Section 6.3.2.1.2 of DOE-STD-3014-2006 (Reference 10.1.11)*." Based on review of the referenced documents of Section 6.3.2.1.2 of DOE-STD-3014-2006, and Section 2.1.2.2 of NEI 07-13, Revision 8, it is not clear to staff whether Equation 6-53 can be applicable for the SC walls.

Request

The staff requests the applicant to describe whether Equation 6-53, in the TR, is applicable for the SC walls.

NuScale Response:

Section 6.7.2 includes the method for the evaluation of a wall with certain concrete and faceplate thicknesses for impactive and impulsive loads. It also provides guidance on the design and sizing of Steel Composite (SC) walls for these loads. For SC wall design, an initial concrete wall thickness has to be assumed and is used to calculate the required faceplate thickness. Although Equation 6-53 in TR-0920-71621 was not developed specifically for SC walls, it provides a reasonable, initial concrete thickness value, which is then utilized in SC wall design equations. Concrete thickness of 70 percent of the reinforced concrete wall thickness given in Equation 6-53 is an initial assumption. Based on this assumed value, the required faceplate thickness to prevent perforation is calculated. The SC wall thickness design can be an iterative process for which the concrete thickness is adjusted to result in an acceptable faceplate thickness. This process is in agreement with the commentary of ANSI/AISC N690-18, Section N9.1.6c.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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

{{

}}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~~or~~ 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, ~~and~~ 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.

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

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 Vp is calculated using the procedure described in Section 2.1.2.4 of NEI 07-13. Bruhl, et al. simplifies the procedure as follows:

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-7

Requirement

10 CFR Part 50, Appendix A, General Design Criteria (GDC) 1, 2 and 4 as it relates to the design of seismic Category I structures, systems and components (SSCs).

DSRS 3.8.4, "Other Seismic Category I Structures," states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by NRC Regulatory Guide (RG) 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

The TR described new developments in the design and evaluation of complex seismic Category I and II structures for the new generation of SMR designs but did not provide the definitions of seismic Category I and II structures.

Request

The staff requests the applicant to provide the definitions of seismic Category I and II structures.

NuScale Response:

The following definitions of Seismic Category I and II structures are added to TR-0920-71621 Table 1-2.

Seismic Category I are structures, systems and components (SSCs) designed to withstand the seismic loads associated with the safe shutdown earthquake (SSE), in combination with other designated loads, without loss of function or pressure integrity.

Seismic Category II are 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.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-8

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.1, "General Information," of the TR, the applicant compared "the element-based approach" and "the member-based or section cut-based approach." The applicant concluded that "the member-based or section cut-based approach" yields design demands that are more realistic based on TR Reference 10.1.27, "Kohli, T., Gurbuz, O., Ostadan, F., "Integrated Seismic Analysis and Design of Shear Wall Structures," Bechtel Technology Journal, 2008."

The staff review of reference 10.1.27 in the TR provided an integrated design approach using information from SASSI2000 (transfer functions) and SAP2000 (dead-loads) that are imported into the "optimum concrete (OPTCOM)" reinforced concrete design proprietary computer code that performs, on a time-step basis for the entire time-history, a "best-fit" reinforcement solution at each concrete slab/wall element in an element grouping. However, it is not clear to the staff how the applicant will use this integrated design approach with SASSI2000 and ANSYS finite element computer codes.

Request

The staff requests the applicant to describe the implementation of this integrated design approach using SASSI2000 and ANSYS finite element computer codes.

NuScale Response:

Reference 10.1.27 1 Kohli, T., Gurbuz, O., Ostadan, F., “Integrated Seismic Analysis and Design of Shear Wall Structures,” Bechtel Technology Journal, 2008, was added to show that both element-based and section cut-based methods have been used to design safety-related structures. TR-0920-71621 does not indicate that the integrated design approach provided in Reference 10.1.27 was used.

The methodology for designing reinforced concrete structures is described in Section 8.0 of TR-0920-71621. NuScale seismic analysis methodology is described in NuScale technical report TR-0118-58005-P-A, Revision 2, Improvements in Frequency Domain Soil-Structure-Fluid Interaction Analysis.

Additional explanation is added to TR-0920-71621 to describe how the cut-section design methodology is implemented.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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. ~~and guidance from Regulatory Guide 1.142 (Reference 10.1.6).~~ Pertinent requirements are also included from Regulatory Guide 1.142 (Reference 10.1.6~~Reference 10.1.7~~).

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).

Response to Request for Additional Information
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Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-9

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.4, "Design Methodology," of the TR, the applicant described the details of section cuts at a slab panel modeled with shell elements and section cuts for columns and T-beams. In Figure 8-3, "Section cut for columns and section cut local coordinate system," the applicant presented the selected finite element as a section cut. It is not clear to the staff how the applicant would consider the effective length (k factor) of a column in the design.

Request

The staff requests the applicant to describe how the effective length (k factor) of a column is considered in the design.

NuScale Response:

The effective length (k factor) of a column is considered in the design as described in TR-0920-71621 Section 8.12.4. For each column, k is conservatively taken as 1 and Equation 8-25 is used to determine whether slenderness effects are to be included in the design. If needed,

slenderness effects are included in design calculations using any of the three available methods specified in ACI 318 Sections 10.10.3, 10.10.4 and 10.10.5 (ACI 349 Section 10.10 directly refers to ACI 318 Section 10.10 and its subsections).

Impact on Topical Report:

There are no impacts to Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, as a result of this response.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-10

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.2, "Purpose," of the TR, the applicant described the objectives of Section 8.0 which is to develop a methodology for the design of Seismic Category I and II structures to the requirements of ACI 349-13 and RG 1.142. The staff compared Table 8-2, "Load Combinations for Concrete Structures," in Section 8.9, "Load Combinations," against Table 1, "Loads, Load Factors and Load Combinations," in RG 1.142, and noted differences, e.g. factors and load combinations.

Request

The staff requests the applicant to provide the justification for the differences between the load combinations in Table 8-2 of the TR and those listed in Table 1 in RG 1.142.

NuScale Response:

Table 1 below provides comparison of the differences, indicated in bold, in load factors between RG 1.142 Table 1 and TR-0920-72621 Table 8-2.

As stated in ACI 349-13 commentary Section R9.2.1 and also discussed in Dr. Bruce Ellingwood's review of BNL-211992-2019-INRE (included as an attachment to AISC Task Committee 11 June 2020 meeting agenda), the load combinations provided in the recent editions of design codes (ACI 349-13, AISC N690-18) are developed using probabilistic concepts by considering the principal and companion action methodology. In the principal and companion action methodology, for a given load combination, the load representing the principal action (e.g. earthquake load, E_{SS}, in load combination 9-6) is taken at its maximum lifetime (peak) value, whereas other loads that are included in the same load combination (e.g. live load, L, with 0.8 load factor) are considered as companion action and taken at their arbitrary point-in-time (mean) value. In the probabilistic combination of loads, the occurrence of peak value for each load is considered to be low probability and, hence, the load factors for loads representing companion action are defined to be lower compared to when those loads represent the principal action. NuScale selected the load factors in ACI 349-13 instead of the revised values in RG 1.142 because the values in ACI 349-13 more closely align with recent design codes for developing load combinations.

The load factor for the live load in ACI 349-13 load combinations 9-6 through 9-9 is 0.8. This value is lower compared to the live load factor of 1.0 defined for the similar set of load combinations (C.9-4 through C.9-8) in Appendix C of ACI 349-13. As discussed in public submission for comments on DG-1283 (ML19176A439), the live load factor defined in Appendix C of ACI 349-13 is higher because the associated load combinations are used with higher strength (phi) factors. The strength factors associated with the load combinations defined in Section 9.2 of ACI 349-13 are lower and, hence, the load factors are revised to maintain a similar level of global safety factor among different versions of design codes. Furthermore, for ACI 349-13 load combination 9-6 through 9-9, live load is considered as a companion action and as discussed in ACI 349-13 commentary Section R9.2.1, the load factor of 0.8 is based on a consensus estimate in NUREG/CR-3315 and represents the arbitrary point-in-time (mean) value of live load.

In ACI 349-13 Section 9.2, the load factors provided for T_o and R_o are lower than the ones defined in Appendix C of the same standard. As discussed in ACI 349-13 commentary section R9.1.1 and R9.2.1, the main reason behind this reduction is that R_o and T_o are considered to have lower variability in estimation compared to other loads such as the live load. However, ACI

349-13 requires the designer to evaluate variability in estimating these loads and, if needed, to increase corresponding load factors accordingly. NuScale plants are designed and evaluated for a minimum and maximum range of operating temperatures and flow transients. On this basis, the variability in the estimate of R_o and T_o is considered to be adequately represented by the load factors defined in load combinations provided in Section 9.2 of ACI 349-13 and assumed that the base load factors need not be increased to the values provided in RG 1.142.

Table 1: Comparison of load factors between RG 1.142 and ACI 349-13

ACI 349-13 Equations	**RG 1.142 Rev. 3 Table 1**	**TR-0920-71621 Table 8-2**
	Normal Load Combinations	
(9-1)	$1.4(D + F) + \boldsymbol{R_o} + T_o$	$1.4(D + F + R_o) + T_o$
(9-2)	$1.2(D + F) + 1.6(L + H \boldsymbol{+ R_o} + \boldsymbol{T_o}) + 1.4C_{cr} + 0.5(L_r \text{ or } S \text{ or } R)$	$1.2(D + F + T_o + R_o) + 1.6(L + H) + 1.4C_{cr} + 0.5(L_r \text{ or } S \text{ or } R)$
(9-3)	$1.2(D + F) + 0.8(L + H \boldsymbol{+ R_o} + \boldsymbol{T_o}) + 1.4C_{cr} + 1.6(L_r \text{ or } S \text{ or } R)$	$1.2(D + F + R_o) + 0.8(L + H) + 1.4C_{cr} + 1.6(L_r \text{ or } S \text{ or } R)$
	Severe Environmental Load Combinations	
(9-4)	$1.2(D + F) + 1.6(L + H \boldsymbol{+ R_o} + E_o) \boldsymbol{+ 1.4C_{cr}}$	$1.2(D + F + R_o) + 1.6(L + H + E_o)$
(9-5)	$1.2(D + F) + 1.6(L + H \boldsymbol{+ R_o} + W)$	$1.2(D + F + R_o) + 1.6(L + H + W)$
	Extreme Environmental Load Combinations	
(9-6)	$D + F + \boldsymbol{1.0L} + C_{cr} + H + T_o + R_o +$	$D + F + 0.8L + C_{cr} + H + T_o + R_o + E_{ss}$

	E_{ss}	
(9-7)	D + F + ***1.0L*** + H + T_o + R_o + W_t	D + F + 0.8L + H + T_o + R_o + (W_t or W_h)
	Abnormal Load Combinations	
(9-8)	D + F + ***1.0L*** + C_{cr} + H + T_a + R_a + ***1.4*P_a**	D + F + 0.8L + C_{cr} + H + (T_a + R_a + 1.2P_a)
(9-9)	D + F + ***1.0L*** + H + T_a + R_a + P_a + Y_r + Y_j + Y_m + E_{ss}	D + F + 0.8L + H + (T_a + R_a + P_a) + (Y_r + Y_j + Y_m) + E_{ss}

Impact on Topical Report:

There are no impacts to Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, as a result of this response.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-11

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.5, "Required Strengths for Slab and Basemat Design," of the TR, the applicant stated "*Conservatively, the length of section cuts is defined to be less than three times the member thickness,*" and "*In this case, section cut lengths are not limited to three times the member thickness.*" In Section 8.5.1, "Section Cuts for Out-of-Plane Forces," the applicant stated, "*Based on recommendations in Reference 10.1.29, the section cut length is limited to three times the basemat thickness.*" In Section 8.5.4 "Determination of Section Cut Locations using FEA Stress Resultants," the applicant stated, "*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.*"

Based on these statements, it is not clear to the staff how the optimal length of the section cuts were determined (e.g. in Section 8.5, the section cut length is less than three times the member thickness which may be conservative, while in Section 8.5.4, the cut length need not to be less than three times the slab thickness.)

Request

The staff requests the applicant to describe the basis for how to interpret Sections 8.5, 8.5.1 and 8.5.4 in determining the most appropriate section cut length.

NuScale Response:

TR-0920-71621 Section 8.5.1 provides general guidelines to determine critical section cut locations for out-of-plane moment and out-of-plane shear demand in slabs. 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. Many of the slabs in the NuScale Seismic Category I and II buildings fit in this simple configuration. 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.

The section cut lengths in Section 8.5.1 of TR-0920-71621 are conservatively limited to three times the member thickness to capture the effect of concentrated loads. Three times the slab thickness is an effective length of slab that is allowed by design codes to resist concentrated loads (e.g., column loads). Additional section cut locations are investigated to account for the effects of non-uniform loads and geometrical irregularities on the location of critical regions in slabs.

These additional section cut locations are identified using the finite element analysis results in the form of stress resultant contour plots, as described in TR-0920-71621 Section 8.5.4. In this approach, the actual stress distribution 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 few finite elements located adjacent to openings or located where finite element mesh or component geometry include abrupt changes. These stress concentrations are not representative as concrete will crack and distribute the localized stresses over a larger area.

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 results 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. It is incumbent upon the design engineer to justify the use of three times or other averaging lengths in all cases.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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.

~~The following report sections~~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~~simple slab configurations~~. 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.~~However, the basic concepts described in the following report sections are intended to be used in the determination of the section cut locations for the design of the floor slabs and foundation basemats of the NuScale buildings.~~

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 ~~the following sections~~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~~are considered as the minimum set of locations for demand extraction and, as needed, additional section cuts are evaluated~~ to investigate the possibility of larger demands outside these critical locations. ~~Conservatively, t~~The length of section cuts, defined in Section 8.5.1, ~~is defined to be less than~~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.~~three times the member thickness.~~

The additional section cut locations ~~and their length can be determined based on the FEA stress resultants~~are identified using finite element analysis results in the form of stress resultant contour plots, as described in Section 8.5.4. ~~In this case, section cut lengths are not limited to three times the member thickness.~~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.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-12

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.5.1, "Section Cuts for Out-of-Plane Forces," of the TR, the applicant stated, "*The SC wall connection to the basemat is designed to sustain 125 percent of the SC wall tensile capacity. The tension demand in the SC walls translate to out-of-plane shear in the basemat. For this reason, the out-of-plane shear demand obtained at the section cut defined in TR Figure 8-9 is increased by 25 percent to account for the overstrength and to ensure ductile behavior in the SC wall-basemat connection.*" It is not clear to the staff why the applicant increased the out-of-plane shear demand by 25 percent to account for the overstrength and to ensure ductile behavior in the SC wall-basemat connection.

Request

The staff requests the applicant to describe the basis to increase the out-of-plane shear demand by 25 percent to account for the overstrength and to ensure ductile behavior in the SC wall-basemat connection.

NuScale Response:

AISC N690-18, Section N9.4.2, states that for the Steel-Plate-Composite (SC) "The required strength for the connections shall be determined as: (a) 125% of the smaller of the corresponding nominal strengths of the connected parts, or (b) 200% of the required strength due to seismic loads plus 100% of the required strength due to nonseismic loads (including thermal loads)".

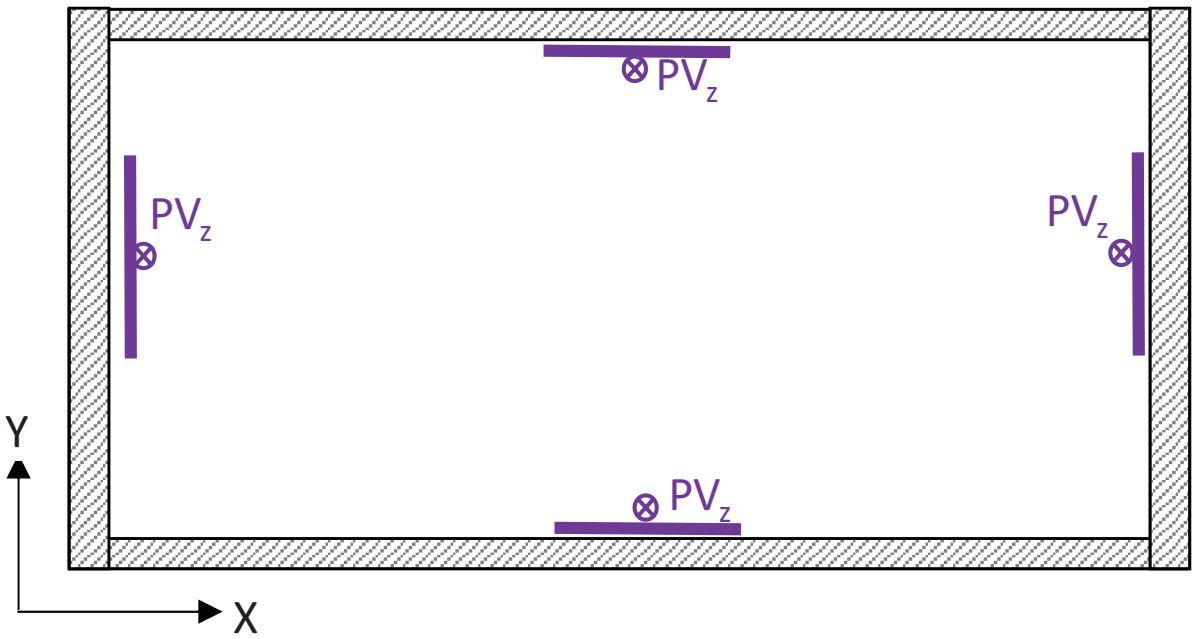
Option (a) is the preferred option as connections designed for this required strength develop the expected capacity of the weaker of the connected parts. TR-0920-71621 Section 7.4 describes multiple SC wall-to-basemat connection configurations. The connections are achieved using either base plate and anchoring rebar or anchoring rebar alone. To achieve a full-strength connection, these elements need to be designed to transfer 125 percent of the smaller of the corresponding nominal strengths of the connected parts. However, the out-of-plane shear demand in the basemat does not need to be increased by 25 percent.

TR-0920-71621 Section 8.5.1 is revised to remove the statement.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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.

~~In agreement with the current design practice (e.g., Reference 10.1.29), the additional section cuts for out-of-plane shear are located at one basemat effective depth, *d*, from the free end of walls imposing overturning moments and extends one basemat thickness, h_b, at either side of the wall, as shown in red in Figure 8-9.~~

~~If the out-of-plane shear demand calculated at these section cuts exceeds the basemat capacity corresponding to one-way or beam shear, then the failure mode would change to two-way or punching shear. In this situation, the shear capacity associated with this failure mode is instead considered and the associated demands are calculated at the section cut shown in red in Figure 8-10.~~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 8-9~~Figure 8-10. Based on recommendations in Reference 10.1.29, the section cut length is limited to three times the basemat thickness.

~~The SC wall connection to the basemat is designed to sustain 125 percent of the SC wall tensile capacity. The tension demand in the SC walls translate to out-of-plane shear in the basemat. For this reason, the out-of-plane shear demand obtained at the section cut defined in Figure 8-9 is increased by 25 percent to account for the overstrength and to ensure ductile behavior in the SC wall basemat connection.~~

Figure 8-8 Out-of-plane forces in foundation basemat due to wall rocking

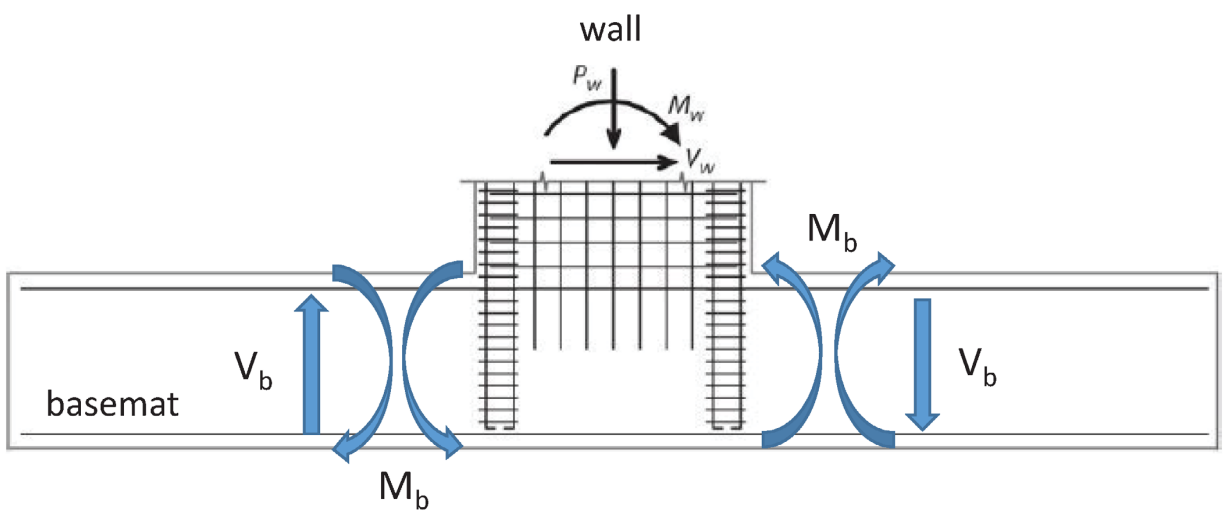


Figure 8-9 Section cuts for one-way and two-way shear in basemat subject to wall overturning~~Section cuts for out-of-plane shear (red) and bending (blue) in basemat subject to wall overturning~~

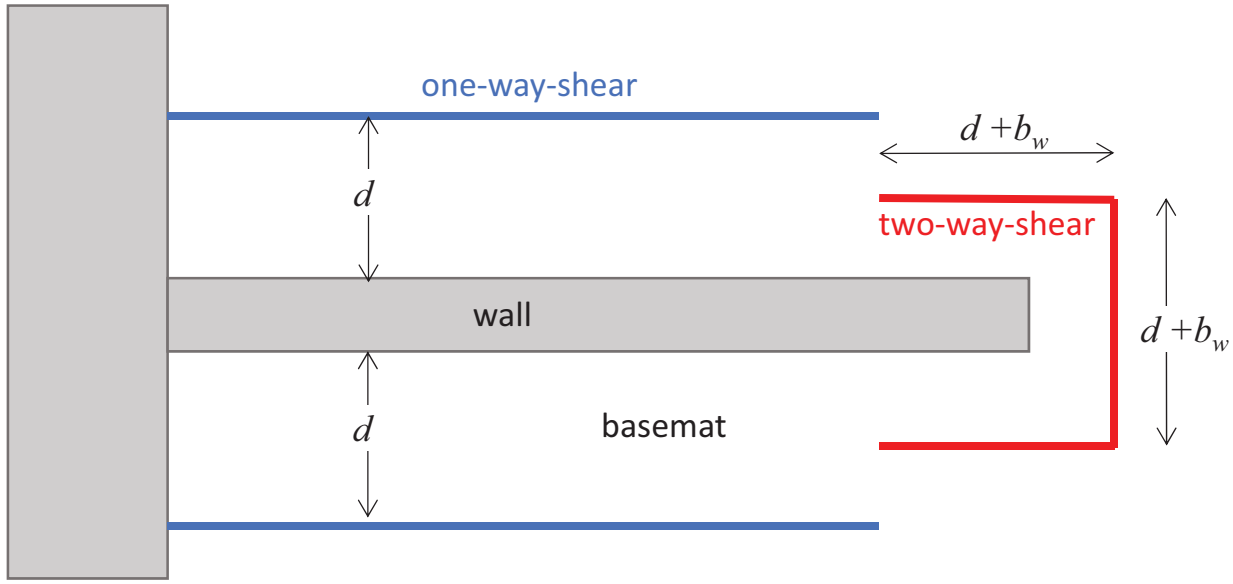
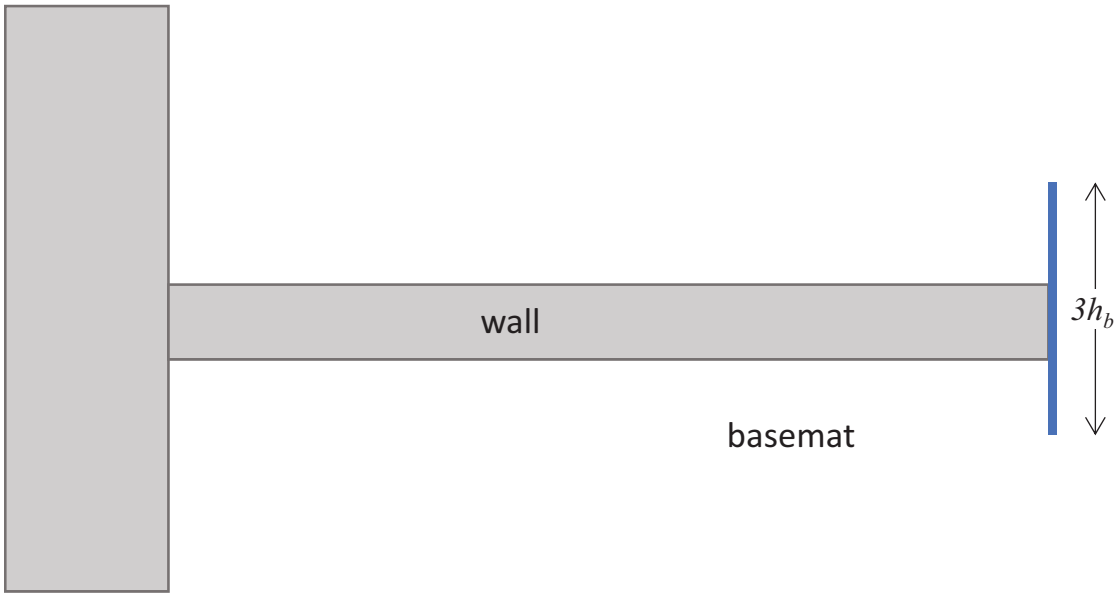


Figure 8-10 Section cuts for out-of-plane bending in basemat subject to wall overturning~~Section cut for two-way shear in basemat subject to wall overturning~~



Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-13

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

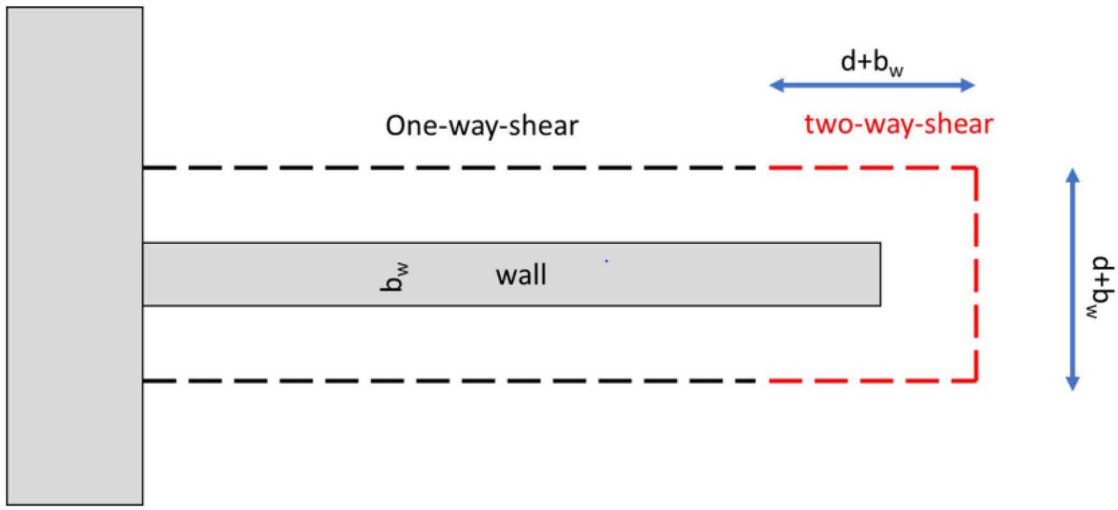
Issue

1. In Figure 8-9 of the TR, the applicant depicted the out-of-plane shear (shown in red) section cut "d" distance (from extreme compression fiber to centroid of longitudinal tension reinforcement) away from the wall. It is not clear to the staff why the applicant chose a distance of "d" for the out-of-plane shear (shown in red) section cut.

2. In Figure 8-10, "Section cut for two-way shear in basemat subject to wall overturning," of the TR, the applicant depicted the square section cut, in red, with a distance of "$d+b_w$," at each side at the end of continuous wall. In Section 8.5.1, "Section Cuts for Out-of-Plane Forces," the applicant also stated, "*if the out-of-plane shear demand calculated at these section cuts exceeds the basemat capacity corresponding to one-way or beam shear, then the failure mode would change to two-way or punching shear. In this situation, the shear capacity associated with this failure mode is instead considered and the associated demands are calculated at the section cut shown in red in Figure 8-10.*"

   However, it is not clear to the staff the source for the change of the failure mode of section

cuts from one-way shear to two-way shear (punching shear), when the out-of-plane shear demand at these section cuts exceeds the basemat capacity corresponding to one-way shear. As shown below, it would be more correct to consider, both one-way shear and two-way shear, to get more accurate shear distribution around the shear perimeter of a slender blade-column in a foundation.



Plan View

Request

a) The staff requests the applicant to describe the basis for choosing the distance "d" for the out-of-plane shear (shown in red) section cut.

b) The staff requests the applicant to describe how the failure mode of cutouts change from one-way shear to two-way shear could independently be evaluated in a slender blade-column in the foundation if the out-of-plane shear demand corresponding to one-way section cuts exceeds the basemat capacity.

NuScale Response:

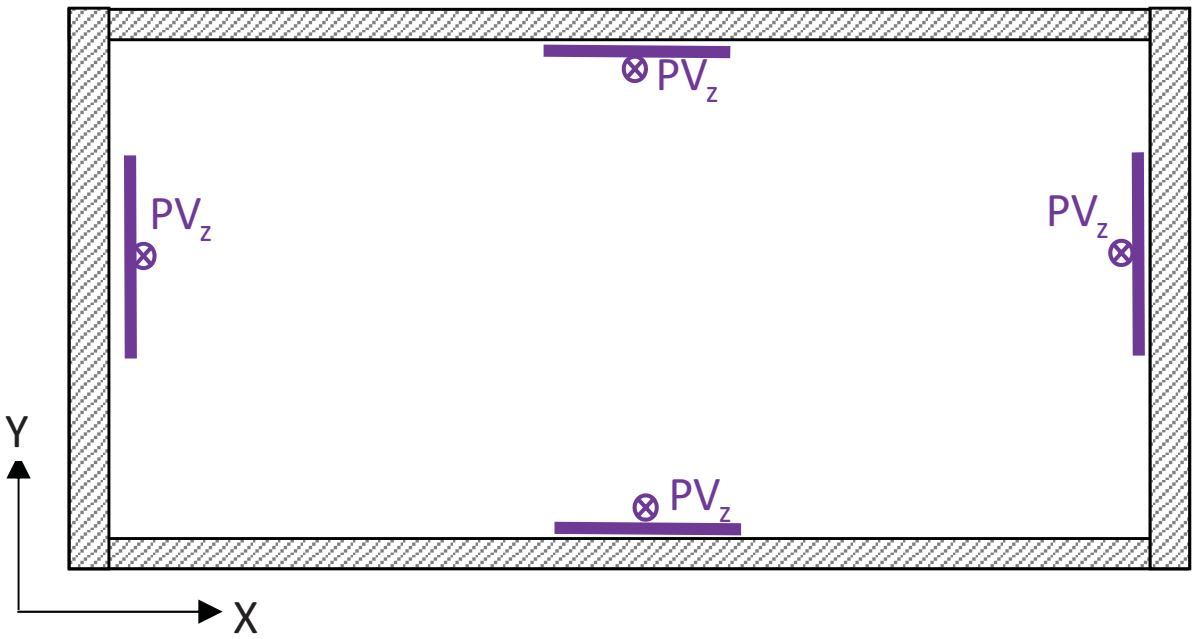
Response Request (a): ACI 318 Section 11.1.3.1 and ACI 349 Section 11.1, which directly references ACI 318 Section 11.1 and its subsections, states that "For nonprestressed members, sections located less than a distance *d* from face of support shall be permitted to be designed for V_u computed at a distance *d*". TR-0920-71621 Figure 8-9 considers the wall to act as support to the basemat for out-of-plane shear, thus, the design demand is calculated at a section cut that is *d* distance from the face of the wall.

Response Request (b): The use of the section cut for one-way shear in TR-0920-71621 Figure 8-9 is a conservative approach to evaluate the out-of-plane shear in the basemat region next to the wall ends. It is included due to its simplicity in calculating the shear demand at this section cut. However, as noted by the Staff, the basemat resists the shear coming from the end regions of the walls through two-way (punching type) shear. Thus, TR-0920-71621 Section 8.5.1 and TR-0920-71621 Figures 8-9 and 8-10 are updated to evaluate both one-way shear along the wall length and two-way shear at the wall ends as recommended by the Staff.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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.

~~In agreement with the current design practice (e.g., Reference 10.1.29), the additional section cuts for out-of-plane shear are located at one basemat effective depth, *d*, from the free end of walls imposing overturning moments and extends one basemat thickness, h_b, at either side of the wall, as shown in red in Figure 8-9.~~

~~If the out-of-plane shear demand calculated at these section cuts exceeds the basemat capacity corresponding to one-way or beam shear, then the failure mode would change to two-way or punching shear. In this situation, the shear capacity associated with this failure mode is instead considered and the associated demands are calculated at the section cut shown in red in Figure 8-10.~~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 8-9~~Figure 8-10. Based on recommendations in Reference 10.1.29, the section cut length is limited to three times the basemat thickness.

~~The SC wall connection to the basemat is designed to sustain 125 percent of the SC wall tensile capacity. The tension demand in the SC walls translate to out-of-plane shear in the basemat. For this reason, the out-of-plane shear demand obtained at the section cut defined in Figure 8-9 is increased by 25 percent to account for the overstrength and to ensure ductile behavior in the SC wall basemat connection.~~

Figure 8-8 Out-of-plane forces in foundation basemat due to wall rocking

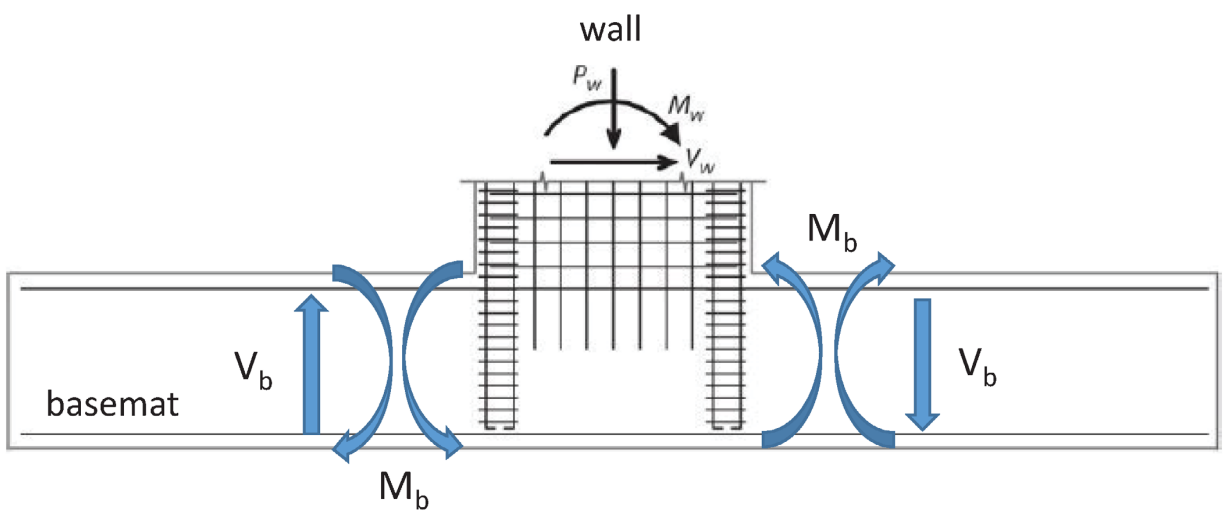


Figure 8-9 Section cuts for one-way and two-way shear in basemat subject to wall overturning~~Section cuts for out-of-plane shear (red) and bending (blue) in basemat subject to wall overturning~~

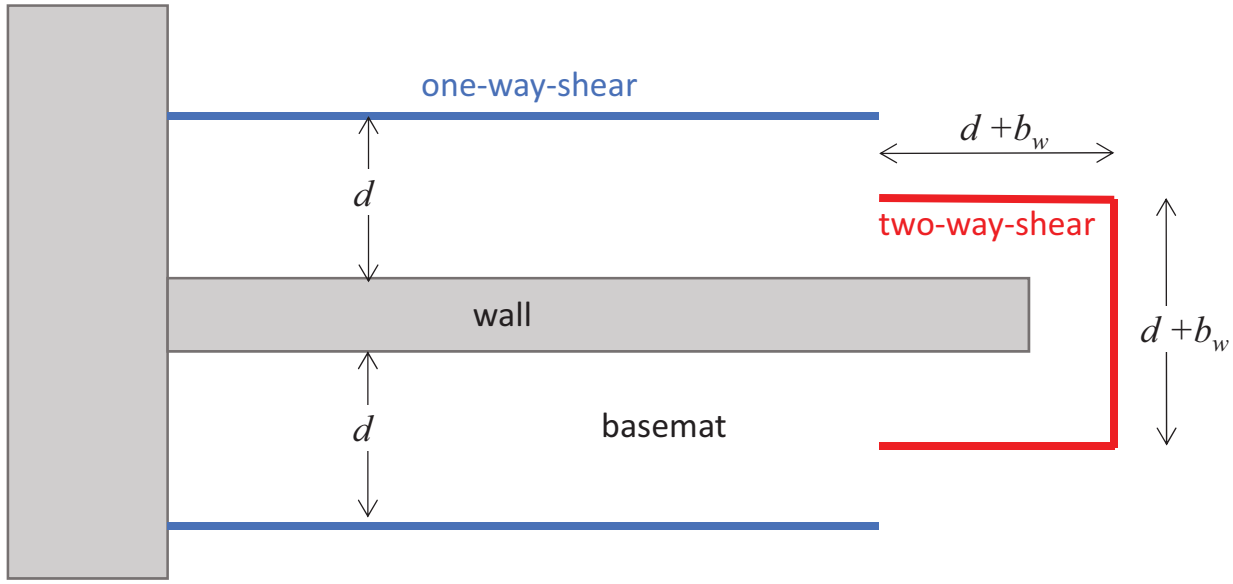
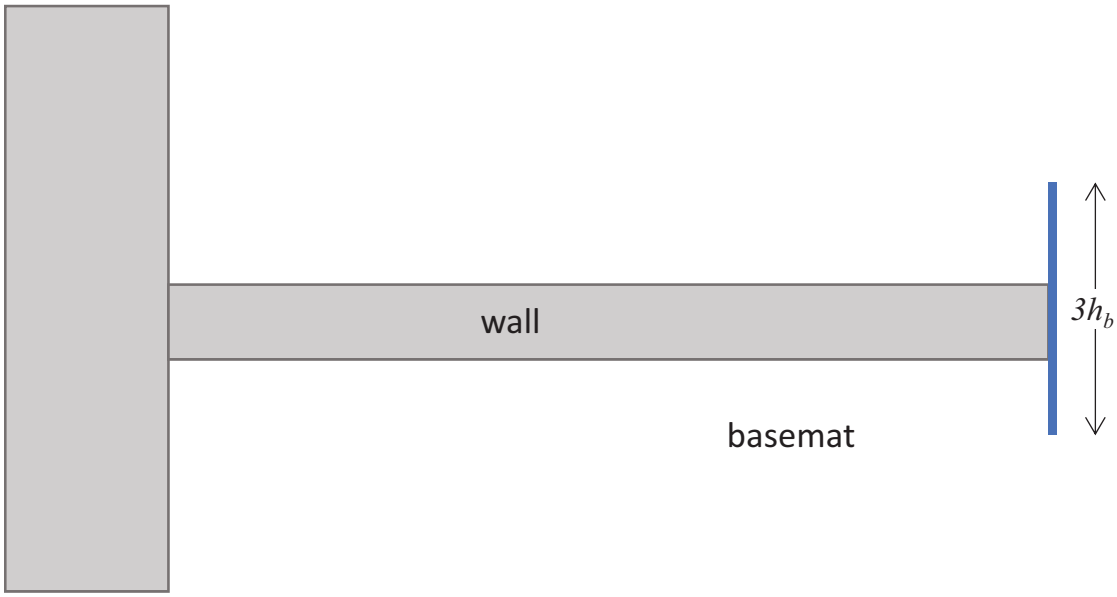


Figure 8-10 Section cuts for out-of-plane bending in basemat subject to wall overturning~~Section cut for two-way shear in basemat subject to wall overturning~~



Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-14

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

SRP 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

{{

}}2(a),(c)

{{

}}2(a),(c)

Request

The staff requests the applicant to explain the basis for incorporating these code provisions into TR Table 8-4.

NuScale Response:

In agreement with ACI 318 Section 7.7.1 Item (c), the minimum concrete cover for slabs is 1-1/2 in., for No. 14 and No. 18 bars, and 3/4 in. for No. 11 bar and smaller. {{

}}2(a),(c)

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

Table 8-4 Minimum specified clear cover for slabs

Member Effective Depth[1]	Slab Thickness[1,2]	Specified Cover[3]	Tolerance on *d*[4] ~~ACI 349 Section 7.5.2.1~~
{{			
			}}[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~~8~~ 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.
~~Where 2 in. clear cover is specified, maximum bar size is No. 8 to satisfy ACI 117-10, Section 2.3.1 with a 1 in. thick embed plate. Where 3 inch clear cover is specified, maximum bar size is No. 11 with up to 1½ inch thick embedment plate. A larger clear cover is needed for thicker embedment 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.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-15

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.10.6, "Bundled Bars for Beams and Columns," of the TR, the applicant provided an equation of $d_{be} = (4\ A_s / \pi)^{1/2}$ that derives a single bar diameter from a unit of bundled bars. It is not clear to the staff from where the applicant obtained that equation.

Request

The staff requests the applicant to provide the source of the above equation.

NuScale Response:

Equation $d_{be} = (4\ A_s / \pi)^{1/2}$ is included in Table A-5 of the American Concrete Institute, “The Reinforced Concrete Design Handbook,” ACI SP-17(14), Farmington Hills, MI, 2015, shown as Reference 10.1.23 in TR-0920-71621.

The d_{be} reflects the diameter of an equivalent bar with area equal to the total area of the bundled bars; i.e., $A_{s\ total} = \pi * (d_{be}^2)/4$.

Cross-reference to Reference 10.1.23 added to the TR-0920-71621 Section 8.10.6.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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:

s_{max} ≤ 16 x longitudinal bar diameter

≤ 48 x tie bar diameter

≤ least dimension of the member

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 $d_{be} = \sqrt{\frac{4A_s}{\pi}}$.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-16

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

a) In Section 8.10.2.1, "Slab and Basemat," of the TR, the applicant tabulated calculated minimum reinforcement ratios for slab and basemat in Table 8-3 for member thicknesses of 24 inches, 36 inches, 60 inches and 96 inches. It is not clear to the staff how the applicant determined the tabulated minimum reinforcement ratios for "ρ other face," and "ρ both faces," in Table 8-3.

b) The applicant provided a formula of $f'_t = f_{ct} = 6.7\ (f_c)^{1/2}$ to determine the tensile strength of concrete. The applicant did not reference the source of this equation for determining the tensile strength of concrete. However, the tensile strength of concrete is about 10 to 15 percent of the compressive strength of concrete, as described in Commentary R10.2.5 of ACI 318-08.

Request

a) The staff requests the applicant to describe how the tabulated minimum reinforcement ratios for "ρ other face" and "ρ both faces," in TR Table 8-3, were determined.

b) The staff requests the applicant to reference the source for the equation determining the tensile strength of concrete.

NuScale Response:

Response Request (a):

TR-0920-71621 Table 8-3 shows the minimum longitudinal reinforcement ratios ρ for slabs to satisfy shrinkage and temperature, as well as flexural requirements. The "ρ tension face" is the minimum reinforcement ratio that should be provided at the tension face for slab sections subjected to flexure (ACI 349 Section 13.3.1).

At the same slab section, when one face is in tension, the other face is in compression. In this case, the "ρ other face" reinforcement ratio corresponds to the additional reinforcement, if any, that should be provided so that the total reinforcement ratio (both faces) is at least the reinforcement ratio for shrinkage and temperature. For members with thickness less than 48 in., the minimum reinforcement ratio is specified in ACI 349 Section 7.12.2.1. For members with thickness of 48 in. or more, the minimum reinforcement ratio is specified as per ACI 349 Section 7.12.2.2, and considering both faces by using No. 11 interior bar spaced at 12 in. The "ρ both faces" column is the minimum reinforcement ratio, considering both faces, when the section is not subject to flexure. In this case, this reinforcement ratio is equal to the total 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. TR-0920-71621 Table 8-3 and Section 8.10.2.1 are revised accordingly.

Response Request (b):

ACI 349 Section R2.1, states that the tensile strength of concrete, f_t', is defined as f_{ct} . ACI 318-08 Section R8.6 defines f_{ct} as 6.7 $(f_c')^{0.5}$, thus $f_t' = f_{ct} = 6.7\ (f_c')^{1/2}$.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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. ~~and guidance from Regulatory Guide 1.142 (Reference 10.1.6).~~ Pertinent requirements are also included from Regulatory Guide 1.142 (Reference 10.1.6~~Reference 10.1.7~~).

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).

8.10.2 Limits on Flexural Reinforcement

8.10.2.1 Slabs and Basemat

~~Concrete slabs and basemats have minimum reinforcement determined based on ACI 349 Section 7.12. At least a minimum reinforcement ratio (to gross concrete area) of 0.0018 is provided for slabs in each direction. Conservatively, the same minimum reinforcement ratio for slabs is used for the foundation basemats.~~

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 ratio (ρ) is summarized in Table 8-3 for the typical slab and basemat thicknesses used in the Seismic Category I and II buildings.~~ 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 ~~slab and basemat~~

Member Thickness (in.)	~~ρ tension face[1]~~	~~ρ other face[2]~~	ρ min[1] ~~both faces[3]~~
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.~~Minimum reinforcement ratio for flexural members (ACI 349 Section 13.3.1)~~

~~[2] Minimum reinforcement ratio for shrinkage and temperature for members with thickness of 48 in. or more (ACI 349 Section 7.12.2.2) and considering No11 interior bar @ 12 in.~~

~~[3] Minimum reinforcement ratio for shrinkage and temperature for members having thickness less than 48 in. (ACI 349 Section 7.12.2.1), or for members with thickness of 48 in. or more (ACI 349 Section 7.12.2.2) and considering No.11 interior bar spaced at 12 in.~~

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.

$$A'_{s,min} = \frac{f_t' A}{f_s''} \leq \frac{A}{100}$$ 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:

$$f_t' = f_{ct} = 6.7\sqrt{f_c'}$$

$$f_s'' = 0.6 f_y$$

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 8-23~~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 8-4~~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)

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-17

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.10.2.2, "T-Beams," of the TR, the applicant stated, "*Based on the requirements specified in ACI 349, Section 10.6.4, the maximum bar spacing for beams is 20 inches.*" The staff could not find the basis for this statement.

Request

The staff requests the applicant to confirm whether the appropriate provision in ACI 349 was referred to for the maximum bar spacing of 20 inches for T-Beams.

NuScale Response:

ACI 349 Equation 10-4 provides the maximum spacing of reinforcement closest to the tension face. From ACI 349 Section 10.6.4, the stress in the reinforcing steel under service loads, f_s, is taken as 40 percent of f_y. The concrete cover, c_c, is assumed as 2 in., the maximum spacing of reinforcement closest to the tension face of beams results in 20 in. This spacing is equal to the maximum spacing allowed in Section 10.6.4 of ACI 349.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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.

~~Based on the requirements specified in ACI 349 Section 10.6.4, the maximum bar spacing for beams is 20 inches.~~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.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-18

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

Equations 8-4 and 8-5 in Section 8.10.2.3, "Columns," of the TR, are not consistent with the corresponding equations in Section 10.9.1 of ACI 318-03.

$A_{st,min} \geq 0.01$ Ag Equation (8-4)
$A_{st,max} \leq 0.08$ Ag Equation (8-5)

Request

The applicant should review the listed equations and may need to modify "the equal signs (=)" to "greater than or equal to sign (≥)" and "less than or equal to sign (≤)", respectively in the TR.

NuScale Response:

ACI 318 Section 10.9.1 states that the "Area of longitudinal reinforcement, A_{st}, for noncomposite compression members shall be not less than 0.01Ag or more than 0.08Ag".

In agreement with ACI 318 Section 10.9.1, TR-0920-71621 Equations (8-4) and (8-5) define the minimum and maximum reinforcement as 0.01Ag and 0.08Ag, respectively. These equations are mathematical equalities defining the lower and upper bounds of the allowable range of reinforcement ratios in columns. TR-920-71621 Equations (8-4) and (8-5) are revised to be consistent with Reference 10.1.16, American Concrete Institute, "Building Code Requirements for Structural Concrete and Commentary," ACI 318-08, Farmington Hills, MI, 2008.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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.

~~Based on the requirements specified in ACI 349 Section 10.6.4, the maximum bar spacing for beams is 20 inches.~~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] ~~ACI 349 Section 7.5.2.1~~
{{			
			}}[2(a),(c)]

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-19

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4, as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4, "Other Seismic Category I Structures," states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis."

Issue

In Section 8.11.6, "In-Plane Shear Transfer through Shear Friction," of the TR, the applicant provided equation of $V_n = (A_y f_y + N_u) \mu$ (Equation 8-19) for the nominal shear-friction capacity. It is not clear to the staff why the applicant added the axial load (N_u) into the equation. Section 11.6.4.1 of ACI 318-08 (Equation 11-25) and Section 21.11.9.3 of ACI-349-13 (Equation 21-10) define the shear friction equations as $V_n = A_y f_y \mu$.

Request

The staff requests the applicant describe why the axial load (N_u) was considered in the Equation 8-19.

NuScale Response:

ACI 318, Section 11.6.7 states that "Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane shall be permitted to

be taken as additive to Av fy, the force in the shear-friction reinforcement...". In order to account for both requirements in a single equation, ACI 318 Equation (11-25) was modified so that the axial load normal to the shear plane, Nu, was included with its respective sign. Thus, when Nu is compression (positive), this force is additive to the force in the shear-friction reinforcement (Av fy). When Nu is tension (negative), this force is subtracted from Av fy, so additional reinforcement is needed for the same level of shear demand.This approach is consistent with Chapter 13.8.3 of "Seismic Design of Reinforced Concrete Buildings", 1st Edition, 2014, by Jack Moehle.

This explanation will be added to TR Section 8.11.6. Also, a note will be added stating that no compression from seismic will be considered in TR Equation 8-19, since these loads are not permanent.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

$$V_n = A_{cv}(2\sqrt{f_c'} + \rho_t f_y) \leq 8A_{cv}\sqrt{f_c'}$$ 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 ϕ = 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:

$$V_n = (A_v f_y + N_u)\mu$$ Equation 8-19

Where:

N_u (lb) is the axial load acting on the shear plane where shear, V_u, is transferred~~corresponding to the shear being transferred (V_u)~~. 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:

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-20

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.12.5, "Collector Capacity," and Section 8.5.2, "t Section Cuts for In-Plane (Diaphragm) Forces," of the TR, the applicant provided information about the collector forces in diaphragms using Figures 8-14 and 8.15. However, the applicant did not describe whether the collector forces are to be considered at the same width (thickness) of the walls (non-continuous) or spread into the diaphragms (slabs).

Request

The staff requests the applicant describe whether the collector forces are to be considered at the same width (thickness) of the walls or spread into the slabs (diaphragms). Furthermore, the applicant is requested to describe how the spread widths are determined if the collector forces were to be considered spreading into the slabs (diaphragms), and whether the collector forces only apply at the base of discontinuous walls.

NuScale Response:

The collector forces are considered to have the same thickness as the walls. As stated in TR-0920-71621 Section 8.5.2, the collector forces are evaluated in diaphragms where the supporting wall is not continuous across the diaphragm depth.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

$$\frac{kl_u}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right) \leq 40 \qquad \text{Equation 8-25}$$

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.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-21

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.14.1, "Slabs and Diaphragms," of the TR, the applicant stated, "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." However, the applicant did not describe how to account for the interaction between in-plane and out-of-plane shears. As it was described in Commentary Section R21.1.1 of ACI 349-13, *"Compression members and slabs and walls under out-of-plane shear loads should be designed for full elastic forces." Further in Section 3, "Principles for Special Structural Wall Design," of NIST GCR 11-917-11, Revision 1, "Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams," states "Although ACI 318 permits factored shear on individual wall segments as high as Vu = 10Φ(f'c)1/2Acv, the flexural ductility capacity for such walls is reduced compared with identical walls having lower shear. This Guide recommends factored shear, calculated considering flexural overstrength (see Section 3.1.3), not exceed approximately 4Φ(f'c)1/2Acv to 6Φ(f'c)1/2Acv so that flexural ductility capacity is not overly compromised."*

Request

The staff requests the applicant describe how to account for the effects of the interaction between in-plane and out-of-plane shears.

NuScale Response:

In agreement with Commentary Section R21.1.1 of ACI 349-13, all structural members in the NuScale SC-I/II structures, including slabs and diaphragms, are designed for full elastic forces; i.e., energy dissipation is not accounted for (F_μ = 1.0) to reduce the earthquake design loads.

Interaction between in-plane and out-of-plane shears is not a well-known or researched phenomenon and is not discussed in ACI 349-13 nor in RG 1.142. It is not applicable and therefore not addressed within TR-0920-71621.

The National Institute of Standards and Technology (NIST) GCR 11-917-11, Revision 1, referenced by the reviewer provides seismic design guidelines for cast-in-plane concrete special structural walls and coupling beams in commercial buildings. As discussed in Section 2.7 of this reference, the seismic design forces in walls are calculated using a force reduction factor, R, which ranges from 5 to 7. This approach results in a wall design where: 1) the lateral load capacity of walls becomes less than the seismic demands from design level earthquake; and, 2) seismic response causes inelastic deformations and structural damage. The inelastic response in walls puts the reinforcing steel in the strain hardening region, which increases its tensile strength above the design yield strength (f_y). This response causes an increase in the flexural capacity of the wall also known as the over strength condition. Since for slender walls (NIST GCR 11-917-11 Section 3.1), the preferred failure mode is flexure, the shear capacity is increased by the flexural over strength so that the wall does not fail in shear. Furthermore, in the inelastic range, the flexural ductility capacity of walls is negatively impacted by the magnitude of the shear demand. In order to minimize this impact, the NIST GCR 11-917-11 recommends a limit to the wall shear demand as specified by ACI 318. The discussions and limitations provided in this paragraph are applicable to structural design approach used in commercial buildings where energy dissipation through inelastic response is utilized to reduce seismic design demands. Such limitations are not applicable to nuclear facilities where lateral seismic demands are resisted by squat shear walls that are designed to remain elastic (F_μ = 1.0) for full seismic design loads.

Impact on Topical Report:

There are no impacts to Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, as a result of this response.

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-22

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

SRP 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.11.1, "One-Way Out-of-Plane Shear Capacity of Slabs and Basemat," of the TR, the applicant described a code provision of "*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,* $Vs > 4 (fc')^{1/2} b_w d$*, the maximum spacing is reduced by one-half.*" The staff noted that the applicant did not refer to the appropriate code provision for this requirement, as: "*(ACI 349 Section 11.4.5.3)*."

Request

The staff requests the applicant to provide the appropriate code provision of "(ACI 349 Section 11.4.5.3)."

NuScale Response:

The code provision of "(ACI 349 Section 11.4.5.3)" is added to TR-920-71621 Section 8.11.1.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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):

$$V_c = \left(2 + \frac{4}{\beta}\right)\sqrt{f_c'}b_o d \qquad \text{Equation 8-12}$$

$$V_c = \left(2 + \frac{\alpha_s d}{\beta}\right)\sqrt{f_c'}b_o d \qquad \text{Equation 8-13}$$

$$V_c = 4\sqrt{f_c'}b_o d \qquad \text{Equation 8-14}$$

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-23

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

Issue

In Section 8.14.1, "Slabs and Diaphragms," of the TR, the applicant tabulated reinforcement ratios in Table 8-6, "Reinforcement design in slabs," at X, Y and Z global directions and clearly depicted X and Y directional reinforcement in Figure 8-25, "Force and moment demand at two perpendicular section cuts in a slab." The applicant tabulated reinforcement ratios in Table 8-6 but did not show Z directional reinforcement (stirrups) in Figure 8-25. In Section 8.10.4 "Out-of-plane Shear Reinforcement for Beams and Slabs," the applicant also provided the applicable code provisions for the out-of-plane reinforcement (stirrups).

Because the arrangements of the out-of-plane reinforcement (stirrups) vary based on the column and slab/diaphragm configurations, the applicant should refer to Section 8.10.4, of TR in Sections 8.14.1 describing the applicable code provisions for the out-of-plane reinforcement (stirrups).

Request

The staff requests the applicant to refer to Section 8.10.4, of TR in Sections 8.14.1 describing the applicable code provisions for the out-of-plane reinforcement (stirrups).

NuScale Response:

TR-0920-71612 Section 8.14.1 is revised to clarify that out-of-plane shear reinforcement for slabs conforms with the reinforcement requirements explained in Section 8.10.4.

Impact on Topical Report:

Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, has been revised as described in the response above and as shown in the markup provided in this response.

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	$\rho_{X,OP,YZ}$		
Total rebar calculation		$\rho_{x,IP,XZ} + \rho_{X,OP,YZ}$	$\rho_{Y,OP,XZ} + \rho_{Y,IP,YZ}$	$\max(\rho_{Y,OP,XZ}, \rho_{Z,OP,YZ})$

Response to Request for Additional Information
Docket: 99902078

RAI No.: 9833
Date of RAI Issue: 05/10/2021

NRC Question No.: NTR-24

Requirement

10 CFR Part 50, Appendix A, GDC 1, 2 and 4 as it relates to the design of seismic Category I SSCs.

DSRS 3.8.4 states the structural acceptance criteria for seismic Category I structures appear in ACI 349, with additional guidance provided by RG 1.142 for concrete structures, AISC N690-1994 for steel structures, and Subsection II.4.J for structures that use modular construction methods evaluated on a case-by-case basis.

SRP 3.8.5, "Foundations," Section II.4.J, states, "Explanation of how loads attributable to construction are evaluated in the design. Some examples of items to be discussed include the excavation sequence and loads from the construction sequence of the mat foundation and walls, as well as the potential for loss of subgrade contact (e.g., because of loss of cement from a mud mat) that may lead to a differential pressure distribution on the mat."

Issue

The applicant did not provide any information related to the evaluations of open configurations of seismic Category I and II structures during the construction sequence phases. The applicant considered the analysis/design forces of members (section cuts) under the completed closed configurations of seismic Category I and II structures. The applicant should also discuss the details for performing additional structural evaluations of the members of seismic Category I and II structures during construction as they are erected in open configurations under the same postulated loadings and load combinations.

Request

The staff requests the applicant to discuss the need to perform additional structural analysis/design evaluations on the members of seismic Category I and II structures as they are erected in open configurations during construction under the same postulated loadings and load combinations.

NuScale Response:

The final excavation and construction sequences are not within the scope of this Topical Report.

Excavation and construction loads are not anticipated to be bounding. This is due to a 25 ft perimeter that will be excavated beyond the reactor building and radioactive waste building footprint, eliminating or minimizing soil pressure on newly-constructed walls and foundations. Additionally, per NuScale DCA Section 3.8.5.6.6, the entire RXB basemat is poured in a very short time. The building is essentially constructed from the bottom up. The main loads (e.g., the reactor pool and the NPMs) are not added until the building is complete. Even then, the weight of an NPM (approximately 1,100 kips buoyant weight) is very small in comparison to the overall weight of the building (600,000 kips, which includes concrete, water, and equipment). Therefore, there are no construction-induced settlement concerns. Settlement analyses have been performed, and settlement-induced effects are considered in the design basis as part of the load combinations. Furthermore, per COL Item 3.8-1, the applicant is responsible for providing monitoring of below-grade walls, groundwater chemistry (if needed), base settlements, and differential displacement.

Impact on Topical Report:

There are no impacts to Topical Report TR-0920-71621, Building Design and Analysis Methodology for Safety-Related Structures, as a result of this response.

Enclosure 3:

Affidavit of Mark Shaver, AF-104578

NuScale Power, LLC

AFFIDAVIT of Mark Shaver

I, Mark 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 Request for Additional Information response reveals distinguishing aspects about the methodology by which NuScale develops its building design and analysis.

   NuScale has performed significant research and evaluation to develop a basis for this 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 response to NRC Request for Additional Information 9833 and 9834. 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 July 16, 2021.

Mark W. Shaver

Mark Shaver