BNL-220652-2020-INRE

Evaluation of the Specification for Safety-Related Steel Structures for Nuclear Facilities, ANSI/AISC N690-18, for Application to Nuclear Power Plants

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December 2020

Nuclear Science and Technology Department

Brookhaven National Laboratory

U.S. Department of Energy

Nuclear Regulatory Commission/Non-NUREG

DISCLAIMER

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ABSTRACT

This report describes the assessment of the recently published ANSI/AISC N690-18, Specification for Safety-Related Steel Structures for Nuclear Facilities. This work was performed for the U.S. Nuclear Regulatory Commission (NRC) for potential endorsement of N690-18 for use in the design of nuclear power plants. Currently the NRC endorses the use of an earlier version: N690-1994 (R2004), and thus, a technical review of the updated N690 Specification, which contains significant changes, is warranted. The N690-18 Specification reflects advances and improvements in steel design gained from experience, analytical studies, and experimental data over the years. The N690-18 specification also contains a new Appendix N9 that addresses the design of safety-related steel-plate composite (SC) walls which are a relatively new type of structural member for which previously there has not been any U.S. code or standard that governs its design.

The technical review of N690-18 was performed in order to assess the adequacy of the provisions for use by the NRC in updating its regulatory guidance for nuclear power plants. The research developed the technical basis for acceptance of the new specification and also identified areas where additional staff guidance is needed for the design of safety-related steel structures at nuclear power plants.

The results of the research have determined that the N690-18 specification is a significant enhancement from the N690-1994 (R2004) edition. The new specification contains substantial upgrades in the design of structural steel and includes new provisions for design of SC wall sections, which can benefit the nuclear power industry as well as the NRC review process. All of the improvements, if endorsed by the NRC with certain regulatory positions, will allow clearer and more consistent regulatory guidance, and will avoid the need for justification by licensees and related reviews each time certain methods or approaches are used.

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EXECUTIVE SUMMARY

Industry codes and standards for nuclear power plants are periodically updated to incorporate advances and improvements in structural design. These updates are made to reflect knowledge gained through experience, analytical studies, and experimental data. They also reflect changes in construction materials, construction practices and changes in design approaches especially those to account for uncertainties. In the case of steel structures, ANSI/IASC N690-18 entitled, “Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities”, was recently published in June 28, 2018.

Nuclear Regulatory Commission (NRC) regulatory guidance for the structural design of safety-related steel structures at nuclear power plants is in NRC NUREG-0800, “Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition.” Currently, the SRP endorses ANSI/AISC N690-1994 (R2004), “Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities.” Since the publication of the N690-1994 (R2004) edition, the specification has been revised several times leading to the current published version of N690-18. N690-2018 differs significantly from N690-1994 (R2004), and thus, there is a need to update NRC guidance on the applicability of N690-18.

The NRC Office of Nuclear Regulatory Research requested Brookhaven National Laboratory (BNL) to perform a research activity to review the updated N690-18 specification to determine its acceptability for endorsement by the NRC. This report describes the work performed by BNL to evaluate the adequacy of the N690-18 specification and provides the technical basis for acceptance of the specification, as well as makes recommendations for any exceptions or additions, for consideration by the NRC in developing their regulatory guidance.

Section 1 of the report presents the background, objective, and scope of the research performed by BNL. Even though N690-18 is applicable to safety-related steel structures at nuclear facilities, the scope for the review in this project is the design, fabrication, and erection of safety-related steel structures in nuclear power plants. N690-18 is not a stand-alone document; it uses the AISC 360-16 “Specification for Structural Steel Buildings,” as the baseline document and simply identifies any additions, deletions, modifications, or replacements to AISC 360-16, to make it applicable to safety-related steel structures in nuclear facilities. Therefore, the review of N690-18 under this research effort also required the review of AISC 360-16.

Section 2 of the report describes the methodology used for the review. BNL reviewed the provisions in N690 and, as needed, the provisions in its referenced AISC 360 specification. This review was done in relation to the NRC requirements and regulatory guidance for the design of safety-related steel structures. BNL also took into consideration for the review the regulatory experience of the NRC staff with recent design certification reviews and BNL’s experience in support of NRC staff license application reviews. Section 2 also describes the chronology of the review performed for different versions of N690. When this project began, the review was performed using the N690-12 specification and the referenced AISC 360-10 specification and evaluated the changes against the currently NRC endorsed N690-1994 (R2004). As subsequent versions of N690 became available, the review effort transitioned to the newer specifications including the latest N690-18 and AISC 360-16 editions.

Section 3 of the report summarizes the general assessment of N690-18. N690-18 and the referenced AISC 360-16 specification are a significant upgrade from the N690-1994 (R2004) edition currently endorsed by the NRC. The current specifications present substantial enhancements in various aspects of the design of structural steel and for the first time provides specific provisions for a new type of structural member referred to as steel-plate composite (SC) wall sections, for which no U.S. code previously existed. A major difference between N690-18 and N690-1994 (R2004) is the use of the load and resistance factor design (LRFD) method or allowable strength design (ASD) method which replaces the allowable stress method contained in the N690-1994 (R2004) edition.

Section 4 of the report provides a Chapter-by-Chapter assessment of N690-18. For most chapters in N690, Section 4 of this report contains evaluations and comparisons of N690-12 and AISC 360-10 to N690-1994 (R2004), and then compares the changes in the current N690-18 and AISC 360-16 to the prior version of N690-12 and AISC 360-10. To assess the adequacy of the updated provisions, BNL relied on (1) comparisons to the original N690-1994 (R2004) edition; (2) recent test data supporting updated provisions; (3) expanded technical information in the Commentaries; and (4) engineering judgement of the technical merits of the updated provisions. Technical concerns were noted when identified.

Section 5 of the report summarizes the conclusions and recommendations resulting from the review. The review concluded that the current version of N690-18, and the referenced AISC 360-16 specification, are a significant enhancement compared to N690-1994 (R2004). The current specifications contain substantial upgrades in the design of structural steel and SC wall sections. If N690-18 is endorsed by the NRC (with applicable regulatory positions), it will allow for clearer and more consistent regulatory guidance, which would benefit the NRC staff and the commercial nuclear power industry. As an example, the use of the LRFD approach would make the overall design approach for safety-related steel structures consistent with the approach in the NRC guidance for the design of safety-related concrete structures. Although BNL recommends that most of N690-18 can be endorsed, Section 5 includes several recommendations for qualification, exception, or addition, for consideration by the staff in developing their regulatory guidance.

ABBREVIATIONS AND ACRONYMS

AHJ	Authority having jurisdiction
AISC	American Institute of Steel Construction
AL	Abnormal loads
ASD	Allowable Strength Design
ASTM	American Society for Testing and Materials
ASME	American Society of Mechanical Engineers
AWS	American Welding Society
BNL	Brookhaven National Laboratory
CMTR	Certified material test reports
DM	Direct second-order analysis method
DOE	Department of Energy
EEL	Extreme environmental loads
ELM	Effective length method
HS	High strength
HVAC	Heating ventillation and air conditioning
IIW	International Institute of Welding
LRFD	Load and Resistance Factor Design
LWR	Light Water Reactor
NL	Normal loads
NRC	Nuclear Regulatory Commission
RC	Reinforced concrete
RG	Regulatory Guide
SC	Steel-plate composite
SEL	Severe environmental loads
SRP	Standard Review Plan
SSCs	Structures, systems and components
SSE	Safe shutdown earthquake

1 INTRODUCTION

1.1 Background

Currently, guidance for design of safety-related steel structures for nuclear power plants is presented in the U.S. Nuclear Regulatory Commission (NRC) NUREG-0800, Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition. Sections 3.8.3, 3.8.4, and 3.8.5 of the SRP refer to the ANSI/AISC N690-1994 (R2004) Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities. Since the publication of N690-1994 (R2004), the specification has been revised several times, with the most current published version being N690-18, dated June 2018.

The 2018 edition of N690 differs significantly from the 1994 (R2004) edition of N690 reflecting advances and improvements in steel design gained from experience, analytical studies, and experimental data. One of the differences between these specifications is the use of the allowable stress design method in N690-1994 (R2004) and the use of either the LRFD method or the ASD method in N690-18. The design of concrete structures other than containments follows an LRFD approach. The use of LRFD approach in N690-18 makes the approach for steel structures consistent with that for concrete structures (with appropriate considerations for each construction material). With the development of new designs for nuclear power plants, there is a need to update the NRC guidance to reflect the newer design codes such as the N690 specification for steel structures.

Another impetus for updating the NRC guidance is the use of steel-plate composite (SC) members in safety-related structures. As shown in Figure 1-1, SC structural members consist of two steel plates spaced apart and filled with concrete between the plates. The faceplates are anchored to the concrete using steel anchors and the two faceplates are connected to each other using steel ties. Such SC members were utilized in several recent nuclear power plant designs. However, no consensus industry code existed in the United States that provided design requirements for these types of members. Previous reviews for reactor applications with SC members required justification by the licensees each time certain methods or approaches were used and required related staff reviews of their adequacy. The new Appendix N9 in N690-18 provides requirements for the design of SC walls.

The NRC anticipates that design submittals for new reactors will use SC construction for some of the safety-related structures. A consensus U.S. standard fully-reviewed by the NRC would support more effective and stable regulatory guidance for the industry and for the staff review of applications and license amendments.

This report is an update to Report BNL-211992-2019-INRE (August 2019). It reflects additional work performed by BNL to finalize its recommendations, based on input by the USNRC/RES Staff, and additional technical information provided by the AISC N690 Committee representatives after August 2019.

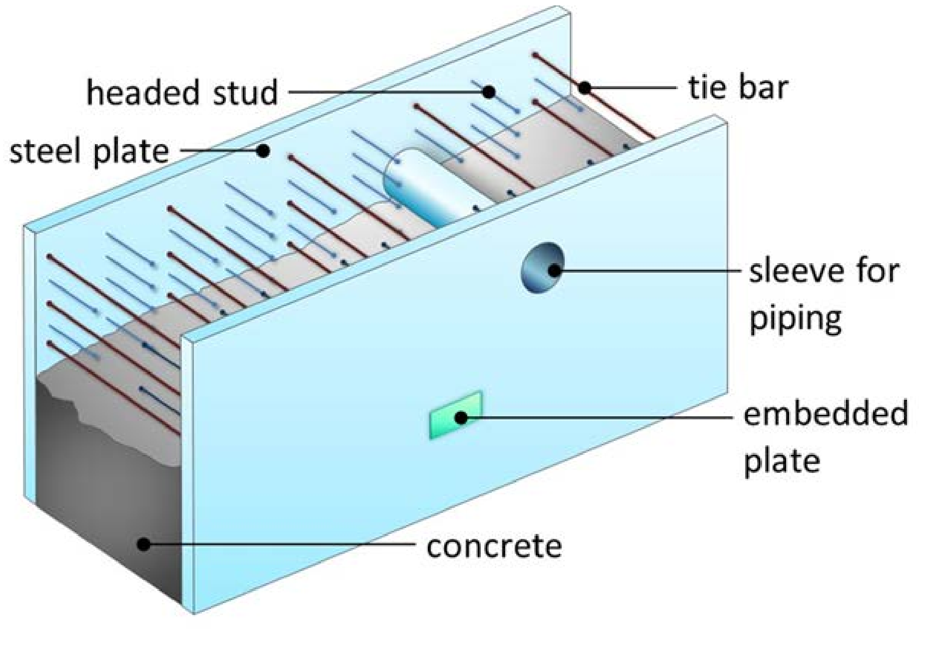

Figure 1-1 Illustration of SC Construction for Walls

1.2 Objective

The objective of this research is to perform a technical review of the updated N690 specification including the new provisions in Appendix N9 of the specification that address the design of safety-related SC walls. The technical review will include an assessment of the sufficiency of the experimental and analytical basis for the standard, particularly for the SC design provisions.

The purpose of this research effort is to provide the technical basis for use by the NRC to develop regulatory guidance, in the form of a new regulatory guide, for the design of safety-related steel and SC structures. The results of this research document the technical basis for the use of N690-18 in NRC staff guidance along with recommendations for any regulatory positions that may be needed for its use.

1.3 Scope

The scope of the research effort is the design, fabrication, and erection of safety-related steel structures for nuclear power plants. Even though the N690 specification is applicable to nuclear facilities, the scope within this project is limited to nuclear power plants.

The N690 specification is not a stand-alone document because it uses the AISC 360-16, Specification for Structural Steel Buildings, as its baseline document (also referred to as the parent standard). The N690-18 specification revises specific parts of AISC 360-16 to make it applicable to safety-related steel structures for nuclear facilities. These revisions consist of additions, deletions, modifications, or replacements to AISC 360-16. Only those sections that differ from AISC 360-16 provisions are indicated in N690-18. Therefore, the review of N690-18, under this research effort, also required the evaluation of the provisions in AISC 360-16.

The scope of N690-18 is safety-related steel structures and structural elements in nuclear facilities. Also included are SC walls (Appendix N9) and steel/concrete composite members (steel beams encased in or filled with concrete, and steel beams supporting a reinforced concrete slab so interconnected that they act together). Specifically excluded from the scope are pressure-retaining components, such as containment, pressure vessels, valves, pumps, piping, and supports for pressure-retaining components whose design follows the provisions of the ASME Boiler and Pressure Vessel Code. Structural steel elements that support safety-related commodities such as cable trays, conduits, HVAC ducts, and instrument lines are within the scope of N690-18. Also within the scope are structural steel elements that support safety-related electrical and mechanical components (beyond the jurisdictional boundary of the ASME component supports where applicable), instruments, and gauges. Further information on the scope of steel structures and structural elements covered by N690 is described in SRP Sections 3.8.3 (Concrete and Steel Internal Structures of Steel or Concrete Containments), 3.8.4 (Other Seismic Category I Structures), and 3.8.5 (Foundations).

2 APPROACH

2.1 Methodology Used for Review

BNL reviewed the provisions of N690-18 and, as needed, the provisions in its referenced (parent) AISC 360-16 specification. This review took into consideration current NRC requirements and regulatory guidance for the design of safety-related steel structures, the regulatory experience of the NRC staff with recent design certification reviews, and reviews of license amendment requests for operating reactors. BNL also used its own experience in support of NRC staff license application reviews and NRC research activities in the review. Of particular importance was the review of the loads and load combinations included in the provisions and the adequacy of the load factors, for consistency with the loads and load combinations specified by the NRC requirements and regulatory guidance.

The approach taken in this research project was to review N690-18 chapter by chapter against the current NRC position on the endorsed version of N690-1994 (R2004). Where the provisions in the N690-18 specification are the same or consistent with those in N690-1994 (R2004), the N690-18 specification is considered to be acceptable. N690-18 relies heavily on its parent specification, AISC 360-16. If N690-18 in a particular chapter or subsection completely replaces the corresponding provision in AISC 360-16, then only the provisions in the N690-18 chapter or subsection was reviewed (i.e., not AISC 360-16). If the chapter or subsection in N690-18 is a modification to the provisions in AISC 360-16 (i.e., addition, deletion, or revision), then both N690-18 and AISC 360-16 were reviewed. If the chapter or subsection in N690-18 indicates no changes are made to AISC 360-16, then only AISC 360-16 was reviewed.

As discussed previously, N690-18 incorporates AISC 360-16 by reference, and identifies additions, deletions, modifications or replacements to AISC 360-16. Use of N690-18 requires the use of AISC 360-16, which is more complex and comprehensive. N690-18 is 196 pages while AISC 360-16 is 676 pages (including the Commentaries). It was not practical for BNL to review every technical detail of N690-18 and AISC 360-16. Such a review was not warranted since both specifications were developed over many years by structural engineers with wide experience and high professional standing and have been confirmed by actual construction experience. Instead, BNL attempted to identify and review significant changes from the previously NRC accepted N690-1994 (R2004) specification. In assessing the acceptability of the changes, BNL relied upon the judgment of its staff and its experience with recent and past licensing reviews, and other activities in support of the NRC. It also solicited opinions of the NRC staff as to issues that they may have encountered with past provisions of the AISC specifications.

During the course of the review, BNL focused its attention more on changes that had the potential to result in significant reductions to the safety margins in the design of steel structures, rather than specific provisions related to structural design details that follow the evolution of structural steel design practice. While the review focused on the provisions in the specifications, BNL also reviewed and relied on the Commentaries to N690-18 and AISC 360-16 to judge the technical merits of the current provisions.

It is noted that the development of the current and prior N690 specifications had representation from the NRC staff on the governing Committee and sub-committee. Thus, the specifications have the benefit of some regulatory perspective and experience in the development of the~~ir~~ provisions.

2.2 Chronology of the Review for Different Versions of N690

In the early stage of this research effort, the review was performed using the N690-12 (2012) specification along with the referenced AISC 360-10 (2010) specification which were the latest available editions at the time this project was initiated. After much of this review had been completed, N690-18 and AISC 360-16 were published. This research effort transitioned to the review of these updated specifications.

There are significant differences in format and content between N690-12/N690-18 and the previously endorsed N690-1994 (R2004). BNL reviewed the evolution of N690-12/N690-18 from earlier specifications. The format of N690-12 and AISC 360-10 more closely follow the first LRFD N690 specification, which was issued in 1986. It appears that many of the newer developments in the design of steel structures were incorporated into the LRFD specification as it was revised through the years from 1986. N690-1994 (R2004) specification focused on the use of the allowable stress design and was last revised only to incorporate Staff technical positions. With issuance of AISC 360-05, the provisions of the LRFD specification and allowable stress design specification were incorporated into one document.[1] It is clear that the bulk of the N690-12 specification was taken from the LRFD specification. Although N690-12 appears different from the N690-1994 (R2004) specification, its roots are based on sound engineering practice and experience that dates back to 1986, and of course earlier specifications as well. In judging the acceptability of N690-12 and N690-18, BNL relied on the fact that many of the provisions are evolutions of previously established provisions from earlier specifications.

The following sections in this report present the results of the reviews of N690-12 and AISC 360-10 against the NRC endorsed N690-1994 (R2004), and the reviews of N690-18 to N690-12 and AISC 360-16 to AISC 360-10.

During the course of the review of N690 and AISC 360, there were two public meetings and one technical exchange meeting held with the ANSI/AISC N690 Task Committee representatives. The public meetings were held on March 19, 2015 and November 23, 2015, and the technical exchange meeting was held on February 26, 2019. All the meetings took place at NRC Offices in Rockville, MD. During these meetings technical information was exchanged to better understand the provisions and commentary in the N690 specification, and the technical basis behind some of the provisions in the specification.

[1] With the publication of AISC 360-05, the allowable stress design in AISC 360 was changed to the allowable strength design (ASD), rather than allowable stress design. With this change, AISC 360 currently defines the allowable strength of a component as its nominal strength divided by a safety factor and requires that its allowable strength exceeds the required strength of the component under the ASD load combinations.

3 GENERAL ASSESSMENT OF N690

N690-18 represents a major revision to N690-1994 (R2004) and incorporates recent developments in the design of conventional steel structures (i.e., AISC 360-16, “Specification for Structural Steel Buildings”) by reference. N690-18 simply identifies additions, deletions, modifications, or replacements to the referenced provisions of AISC 360-16. AISC 360-16 contains detailed specifications for steel structures (conventional and commercial) and incorporates the recent research developments and technology updates in the construction industry. With this approach, the AISC only needed to include in N690-18 those special provisions that are needed to address safety requirements specific to nuclear facilities. By reference to AISC 360-16, N690-18 incorporates both LRFD and ASD methods; detailed requirements for addressing stability of steel structures; guidance for and limitations on the use of advanced analysis methods (nonlinear elastic and inelastic analysis); and updated member design provisions, compared to N690-1994 (R2004). By necessity, the review of N690-18 was also a de facto review of AISC 360-16.

N690-1994 (R2004) differs significantly from the current N690-18 edition. Specifically, they differ in their main approach to design. N690-1994 (R2004) follows an allowable stress design approach while N690-18 follows an LRFD approach, with an option for an ASD (allowable strength design) approach. The LRFD approach is the modern approach for structural design, and is the only approach used for reinforced concrete design of nuclear facilities (ACI-349). It explicitly addresses the uncertainties in the loads (demands) and the resistances (capacities) through specification of separate load and resistance factors of safety. Most research on capacities of steel and steel composite structures and on the definition of demands is now done within the context of LRFD approaches.

N690-1994 (R2004) is a stand-alone document, relatively short and simple to use. The provisions are based on prior AISC steel design specifications that go back many years and are well understood; i.e., allowable stress design. The Commentary is generally adequate to assist the user in implementation.

N690-18, along with the referenced AISC 360-16 specification, is a significant upgrade from the N690-1994 (R2004) edition. The current specification presents substantial enhancements in various aspects of the design of structural steel and for the first time includes specific provisions covering SC wall sections, for which no U.S. Code previously existed.

The review of N690-18 and the referenced AISC 360-16 specification~~,~~ identified a number of specific technical issues that warrant supplementary Staff guidance for application to nuclear power plant steel structures and SC walls. These are documented in detail in Sections 4 and 5 of this report. It was not the intent to assess every change that has occurred over the years, between N690-1994 (R2004) and N690-18. The new specification represents the current thinking and development in the design of steel and SC structures and is acceptable, subject to specific Staff positions recommended by BNL in Section 5 of this report.

4 SPECIFIC ASSESSMENTS OF CHAPTERS/APPENDICES

The following sections provide a Chapter-by-Chapter summary of the observations and assessments by BNL during the review of N690-18.

At the time this project was initiated in 2014, the latest edition was N690-12. The review initially focused on comparing N690-12 to N690-1994 (R2004). N690-1994 (R2004) is the current version accepted by the NRC staff. By the time most of the review was completed for N690-12, AISC was already working on the next revision, scheduled for publication in 2018. BNL reviewed several draft revisions, and finally the official N690-18. Consequently, in most of the sections below, there is first a description of the comparison of N690-12 (and sections of AISC 360-10 incorporated by reference) to N690-1994(2004); this is followed by the comparison of N690-18 to N690-12. Also, since N690-18 incorporated by reference sections of AISC 360-16, the review also included comparison of AISC 360-16 to AISC 360-10.

4.1 Symbols and Glossary

4.1.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

BNL did not trace back to the symbols and definitions used in the N690-1994 (R2004) since this would be a labor-intensive task without much technical benefit. Instead, during the review of the individual chapters, BNL identified any technically significant differences in the symbols and definitions used.

4.1.2 N690-18 Comparison to N690-2012

No technically significant differences in the symbols and definitions used were identified.

4.1.3 AISC 360-16 Comparison to AISC 360-10

No technically significant differences in the symbols and definitions used were identified.

4.2 Chapter NA General Provisions

4.2.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

N690-12 Section NA1 “Scope”, replaces Section A1 of AISC 360-10. It states that the Specification applies to “design” and does not mention fabrication and erection. It also does not indicate it applies to composite structures. N690-1994 (R2004) included fabrication and erection, as well as composite structures (not SC walls). Section A1 of ANSI/AISC 360-10 also includes these items. While the Preface to N690-18 does indicate that the specification addresses the design, fabrication and erection of safety-related steel structures for nuclear facilities, Section NA1, which is labeled “Scope”, should be clarified, indicating that it applies to the design, fabrication, and erection of safety-related steel and composite structures.

N690-12 Section NA1 specifically excludes pressure retaining components (pressure vessels, valves, pumps and piping). N690-1994 (R2004) also has this exclusion. The Commentary to Section NA1 directs the reader to ASME Section III, Subsection NF for plate and shell component supports. This implies that N690-12 covers linear component supports.

N690-12 Section NA1 states that the AISC Seismic Provisions (ANSI/AISC 341), in general, are not applicable. However, it states that the detailing requirements of Sections A3 and D2 “shall be appropriately considered when designing for inelastic behavior.” It may be very appropriate to consider these detailing requirements to incorporate margin to withstand beyond design basis seismic loads, but only elastic behavior limits are acceptable for resisting design basis seismic loads See review of Appendix N1.

Section Q1.0.2 Definitions in N690-1994 (R2004) does not appear in Chapter NA. Apparently these definitions appear in the new standard when the terms are introduced in applicable sections. N690-12 Section NA2, Referenced Specifications, Codes and Standards, adds many references to AISC 360-10 Section A2, and deletes the reference in AISC 360-10 to N690-1994 (R2004). NA2 adds several new references that did not appear in N690-1994 (R2004) and some references from N690-1994 (R2004) were omitted. NA2 is an update of the references to current practice and does not merit further review to track down why each change was made from earlier revisions of this standard.

N690-12 Section NA3, Material, replaces AISC 360-10 Section A3. This section is similar to Section Q1.4 of N690 1994 (R2004) with reformatting and updating. New ASTM standards are added. No issues were identified.

N690-12 Section NA4, Structural Design Drawings and Specifications, replaces AISC 360-10 Section A4. This section relies on reference to Section 3 of the Code of Standard Practice. It includes the same list of items to be included in the construction specification as in Section Q1.1 of N690 1994 (R2004). Section Q1.1 had more information which is probably now covered by reference to the Code of Standard Practice. No issues were identified in this section.

N690-12 Section NA5, Quality Assurance, is new; there is no Section A5 in AISC 360-10. NA5 discusses Appendix B and NQA-1. This Section specifically references the use of NQA-1, requirement 3, for calculations pertinent to the design and NQA-1, Subpart 2.7, for computer programs used in analysis and design. It is not clear why reference is made to just these parts of NQA-1. See further discussion under Chapter NN in this report.

It is recommended that NRC QA staff review N690-12 Section NA5 and Chapter NN for consistency with current staff guidance.

4.2.2 N690-18 Comparison to N690-2012

The following items summarize the changes in Chapter NA

1. N690-18 NA1 added reference to N690-18 Appendix N9.

2. N690-12 NA1 stated the following:

   In the design of members and connections of seismic force resisting systems, the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341), hereafter referred to as the *Seismic Provisions,* in general, are not applicable. However, the detailing requirements of Sections A3 and D2 of the *Seismic Provisions* shall be appropriately considered when designing for inelastic behavior.

   N690-18 NA1 now states the following:

When designing for inelastic behavior such as that caused by impact loads, the design shall follow the AISC 341 material requirements of Section A3 and the general member and connection requirements of Sections D1 and D2 for highly ductile members, respectively.

3. N690-12 NA1 stated the following:

   The sponsors of any structural system or construction within the scope of the Nuclear Specification, the adequacy of which has been shown by successful use or by analysis or test, but which does not conform to or is not covered by the Nuclear Specification, shall have the right to present the data on which their design is based to the *authority having jurisdiction* (AHJ) for review and approval.

   N690-18 NA1 now states the following:

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

4. There were additions to the referenced documents in NA2.

5. The first paragraph of N690-12 NA3.1 stated:

   In addition to satisfying the appropriate ASTM Standards, the specification of the material of those structures or structural components that are subject to suddenly applied dynamic loads (for example, *jet shields, pipe whip restraint,* or *pipe whip impact barriers*) shall be supplemented by the requirement that the material be subjected to Charpy V-notch (CVN) impact tests, using the procedures described in ASTM A20/A20M.

   This was revised in N690-18 NA3.1 as follows:

   In addition to satisfying the applicable ASTM Standards, the specification of the material of those structures or structural components that are subject to impactive and/or impulsive loads shall be supplemented by the requirement that the material be subjected to Charpy V-notch (CVN) impact tests, using the procedures described in ASTM A20/A20M.

6. Two User Notes were added to N690-18 NA3.1.

7. Materials were added to N690-18 NA3.1a, a paragraph was revised and three User's Notes were added.

8. The following paragraph was introduced in N690-18 NA3.1c and NA3.1d:

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

   It replaces the following paragraph in N690-12 NA3.1c and NA3.1d:
   "The project specification covering material for structural components which, as a

result of proposed welding procedures, design details, etc., are susceptible to lamellar tearing shall, as determined by the engineer of record, include the requirement that the material shall be either ultrasonically examined in accordance with ASTM A578/ A578M, Level C or tested in tension in the through-thickness direction (z-direction). The resulting percentage reduction in area in the z-direction shall not be less than 90% of that in the direction of material rolling."

N690-12 NA3.1c and NA3.1d are consistent with N690-94, Section Q1.4.1, which states: "The specification covering material for structural components which, as a result of proposed welding procedures, design details, etc., may be subject to lamellar tearing (where, as determined by the engineer, welding high heat input and/or high restraint can exist), may include the requirement that the material shall be ultrasonically examined in accordance with ASTM A435 and, in addition, may be tested in tension in the through-thickness direction (Z-direction). The acceptance criteria shall be that the resulting percentage reduction in area in the Z-direction shall be not less than 90 percent of the resulting percentage reduction in area in the direction of material rolling when the material is tested in tension in the direction of material rolling."

BNL recommends that the Provisions NA3.1c and NA3.1d in ANSI/AISC N690-18 be qualified that, unless otherwise justified, they should include the ultrasonic examination or testing requirements in NA31.c and NA3.1d of N690-12.

9. N690-18 NA3.7, Material Certification, was added as follows:

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

   This provision replaced the following provision which previously appeared in six places in N690-12 NA3:

   CMTR or certified reports of tests made by the fabricator or a testing laboratory shall verify that the material meets the ASTM specification.

   In lieu of the above, the material supplier or fabricator shall, if approved by the owner and the AHJ, provide a certificate of compliance stating that the steel furnished has been tested and conforms to the ASTM specification.

10. A User Note was added to NA4.

In summary, all changes to Chapter NA, between N690-12 and N690-18, are considered acceptable, with the exception of lamellar tearing.

<u>Evaluation of Commentary to N690-18 Chapter NA</u>

The Commentary was updated to reference recently issued standards. The following discussion highlights the other significant changes to the Commentary.

N690-18 Commentary NA1 "Scope" states the following:

> The AISC *Seismic Provisions for Structural Steel Buildings* (AISC,2016b), hereafter referred to as the *Seismic Provisions*, is intended for the design and construction of steel members and connections in the seismic force resisting systems in buildings for which

> the required strengths resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response.
>
> The requirements of *Seismic Provisions* Sections A3, D1 and D2 are recommended for members and connections subject to localized inelastic response due to the action of certain load actions (such as impact loads, for which local inelastic response is considered acceptable). Conformance with the cited *Seismic Provisions* sections will help the affected members and connections to withstand the load effects without overcoming their force or deformation capacities, as applicable.

The N690-12 Commentary had the same first paragraph, but the second paragraph previously stated:

> The required strengths of seismic force resisting systems in safety-related structures for nuclear facilities are determined from elastic analyses where energy dissipation in the inelastic range is neglected. Thus, in general, the *Seismic Provisions* are not applicable to the design of safety-related structures for nuclear facilities. However, the detailing requirements of Section A3 and Chapter D of the *Seismic Provisions* should be appropriately considered when designing for plastic analysis.

The deleted Commentary paragraph clearly reflects the current Staff position for seismic design of safety-related structures for commercial nuclear power plants. The acceptable use of the Specification for inelastic analysis is reviewed under Appendix N1.

In N690-18 Commentary NA3.1a, the caution on the use of ASTM 167 plate material has been omitted since this material is no longer permitted in NA3.1a of the Specification.

The following was added to N690-18 Commentary NA3.2:

> 2. Steel Castings and Forgings
>
> *Delete the following:*
>
> Design and fabrication of cast and forged steel components are not covered in the Specification.

This change is appropriate since the current and previous issue of N690 covered these materials.

4.2.3 AISC 360-16 Comparison to AISC 360-10

AISC 360-16 adopted an ASTM umbrella bolt specification, ASTM F3125, that includes Grades A325, A325M, A490, A490M, F1852 and F2280.

The following user note appears in the standard:

> User Note: ASTM F3125 is an umbrella standard that incorporates Grades A325, A325M, A490, A490M, F1852 and F2280, which were previously separate standards.

Section A3.3 of the Commentary explains the change as follows:

> ASTM F3125 is an umbrella specification that covers what were ASTM A325/ A325M, A490/A490M, F1852, and F2280 fasteners. These previously separate standards have been unified, coordinated, and made consistent with each other, turning them into Grades of ASTM F3125. From the user perspective, not much has changed, as the head marks remain the same, and handling and installation remain the same. Nevertheless, the specifier should be aware that ASTM F3125 now contains Grade A325, A325M, A490, A490M, F1852 and F2280 fasteners. One change of note is that under F3125, Grade A325 and A325M fasteners are uniformly 120 ksi (830 MPa); Grade A325 and A325M had a drop in strength to 105 ksi (725 MPa) for diameters over one inch (25 mm) in previous standards.

Sufficient justification is provided for this change.

Chapter A – General Provisions - Commentary A1 states:

The scope of this Specification is essentially the same as the 2010 *Specification for Structural Steel Buildings* (AISC, 2010) that it replaces.

There were other modifications and updating to the Specification and Commentary; however, based on a cursory review, a more detailed review is not warranted.

4.3 Chapter NB Design Requirements

For Chapter NB, the provisions of N690-18 and the referenced sections of AISC 360-16 are compared directly to N690-1994 (R2004). This provides greater clarity for presentation of the review findings. It is noted that the preamble to N690-18 Chapter NB is the same as the preamble to AISC 360-16 Chapter B, except for replacing the phrase “design of steel structures” in AISC 360 with “analysis and design of steel structures”; the insertion of the word “Nuclear” in front of “Specification”; and adding the letter “N” in front of the section numbers.

4.3.1 NB1 General Provisions

There is no Section NB1 listed in the N690-18 Table of Contents. Section B1 of AISC 360-16 is “General Provisions.” It only has one sentence indicating that the design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis. Since there are no changes identified in N690-18, it is assumed that AISC 360-16 Section B1 was accepted. This section serves as the replacement for N690-1994, Section Q1.2, Types of Construction. The evolution of the changes is adequately described in the Commentary to AISC 360-16 Section B1.

4.3.2 NB2 Loads and Load Combinations

N690-18 Section NB2, Loads and Load Combinations, replaces AISC 360-16 Section B2 in its entirety, and includes extensive information. This was necessary because AISC 360-16 Section B2 does not identify the specific loads and load combinations. It only indicates that the loads and load combinations shall be those stipulated by the applicable building code, and in the absence of a building code it references ASCE/SEI 7.

N690-18 Section NB2 defines normal loads (NL), severe environmental loads (SEL), extreme environmental loads (EEL) and abnormal loads (AL). The next two subsections in N690-18 define the load combinations for the LRFD approach and for the ASD approach. According to the N690-18 Commentary, the pertinent load combinations in Section NB2 come from ACI 349 (2013), SRP 3.8.3 and 3.8.4 (2013) and RG 1.142 (2001).

An evaluation of the N690-18 LRFD and ASD approaches is summarized below and recommendations for some revisions are proposed. Further details and the basis for the recommendations are described more fully in the Attachment to this report.

Load and Resistance Factor Design (LRFD)

LRFD load combinations are defined in Section NB2.5 of N690-18. The LRFD load combinations did not exist in N690-1994 (R2004) and so they had not been reviewed and endorsed previously by the NRC for design of nuclear safety-related steel structures. If a comparison is made of the LRFD load combinations with the N690-1994 (R2004) load combinations, there are a number of differences which should be expected based on the different approach between the LRFD and the ASD methods. As explained in the Commentary to N690-18, "The load combinations [*for the LRFD approach*] stem from a probability-based study of load combinations for design of nuclear power plants (Hwang et al., 1987). The probabilistic methodology in that study is consistent with that used to develop the probability-based load combination requirements appearing in ASCE/SEI 7-16 (ASCE, 2016), Galambos et al. (1982), and Ellingwood et al. (1982)."

Comparison of N690-18 and ACI 349-13 Load Combinations

Since ACI 349-13, applicable to reinforced concrete structures, uses the LRFD method as does N690-18, it is useful to make a comparison of N690-18 with ACI 349-13. Table 1 provides the LRFD load combinations in N690-18 and ACI-349-13.

Table 1 - LRFD load combinations in AISC N690-18 and ACI 349-13*

AISC N690-18 Section NB2 LRFD		Load, Eqn. #
U = 1.4(D + F) + (1.4Ro + To)	+ C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.2Ro + 1.2To)	+ 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + 1.2Ro + 1.2To)	+ 1.4C + 1.6(Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + W	+ 1.0C + 0.5(Lr or S or R)	SEL, (NB2-4)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + 1.6Eo	+ 1.0C + 0.2(Lr or S or R)	SEL, (NB2-5)
U = D + F + 0.8L + H + To + Ro + Es	+ C	EEL, (NB9-6)
U = D + F + 0.8L + H + To + Ro + Wt		EEL, (NB9-7)
U = D + F + 0.8L + H + Ta + Ra + 1.2Pa	+ C	AL, (NB9-8)
U = D + F + 0.8L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es		AL, (NB9-9)

ACI 349-13 Section 9.2.1 LRFD	Load, Eqn. #
U = 1.4(D + F) + (1.4Ro + To)	NL, (9-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.2Ro + 1.2To) + 1.4C + 0.5(Lr or S or R)	NL, (9-2)
U = 1.2(D + F) + (0.8L + 0.8H + 1.2Ro + **0.0To**) + 1.4C + 1.6(Lr or S or R)	NL, (9-3)
U = 1.2(D + F) + (**1.6**L + 1.6H + 1.2Ro + **0.0To**) + **1.6**W + **0.0C + 0.0(Lr or S or R)**	SEL, (9-5)
U = 1.2(D + F) + (**1.6**L + 1.6H + 1.2Ro + **0.0To**) + 1.6Eo + **0.0C + 0.0(Lr or S or R)**	SEL, (9-4)
U = D + F + 0.8L + H + T_o + Ro + Es + C	EEL, (9-6)
U = D + F + 0.8L + H + To + Ro + Wt	EEL, (9-7)
U = D + F + 0.8L + H + Ta + Ra + 1.2Pa + C	AL, (9-8)
U = D + F + 0.8L + H + Ta + Ra + Pa + Yr + Yj + Ym + **1.0**Es	AL, (9-9)

*Footnotes and caveats applicable to this table are given in the Attachment to this report.

The bold type font in this table highlights the differences between the N690-18 LRFD load combinations and the load combinations in Chapter 9 of ACI 349-13. For wind load (W), N690 uses a load factor of 1.0 for W while ACI uses a factor of 1.6. The 1.6 in ACI is based on a design wind speed for a 100-year return period recommended in ASCE 7 prior to 2010. The factor of 1.0 for W in N690 is associated with a design wind speed for a 3000-year return period for risk category IV, which is the wind treatment in ASCE 7-16. The ratio of the nominal wind speeds using the ASCE 7 prior to 2010 and the ASCE 7-16 varies somewhat depending on the site conditions but is about 1.6, which is the difference in the load factors. Explanation for the other changes shown in bold for the ACI 349 load combinations is presented in the Attachment to this report.

Proposed LRFD Load Combinations for the DG endorsing N690-18

Table 2 provides the LRFD load combinations in N690-18 and in RG 142 Revision 3, followed by the proposed guidance for the load combination in N690-18.

RG 1.142 Revision 3 found the load combinations in ACI 349-13 acceptable for the design of safety-related reinforced concrete structures other than reactor vessels and containments with a few exceptions and additions. ACI 349-13 and RG 1.142 use a LRFD approach for design of safety-related concrete structures. Although it may appear desirable to use the same load combinations for reinforced concrete structures and the structures in the scope of N690, steel structures, composite structures and SC walls, differences exist on the significance of the uncertainties in a few loads for the response of structures in the scope of ACI 349-13 and AISC N690-18. In addition, new understanding of load combinations and new definition of nominal loads should also be accounted for in the proposed load combinations.

Table 2 - LRFD load combinations in N690-18, RG 142 and proposed for this DG*

AISC N690-18 Section NB2 LRFD	**Load, Eqn. #**
U = 1.4(D + F) + (1.4Ro + To) + C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.2Ro + 1.2To) + 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + 1.2Ro + 1.2To) + 1.4C + 1.6(Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + 1.0W + 1.0C + 0.5(Lr or S or R)	SEL, (NB 2-4)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + 1.6Eo + 1.0C + 0.2(Lr or S or R)	SEL, (NB 2-5)
U = D + F + 0.8L + H + To + Ro + Es + C	EEL, (NB 9-6)
U = D + F + 0.8L + H + To + Ro + Wt	EEL, (NB 9-7)
U = D + F + 0.8L + H + Ta + Ra + 1.2Pa + C	AL, (NB 9-8)
U = D + F + 0.8L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es	AL, (NB 9-9)

RG 1.142 LRFD	**Load, Eqn. #**
U = 1.4(D + F) + (1.0Ro + To) (No C load here)	NL, (9-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.6Ro + 1.6To) + 1.4C + 0.5(Lr or S or R)	NL, (9-2)
U = 1.2(D + F) + (0.8L + 0.8H + 0.8Ro + 0.8To) + 1.4C + 1.6(Lr or S or R)	NL, (9-3)
U = 1.2(D + F) + (1.6L + 1.6H + 1.6Ro) + 1.6W (No C or roof loads)	SEL, (9-5)
U = 1.2(D + F) + (1.6L + 1.6H + 1.6Ro) + 1.6Eo + 1.4C (No roof loads)	SEL, (9-4)
U = D + F + 1.0L + H + T_o + Ro + Es + C	EEL, (9-6)
U = D + F + 1.0L + H + To + Ro + Wt	EEL, (9-7)
U = D + F + 1.0L + H + Ta + Ra + 1.4Pa + C	AL, (9-8)
U = D + F + 1.0L + H + Ta + Ra + Pa + Yr + Yj + Ym + 1.0Es	AL, (9-9)

Proposed Guidance for DG LRFD	**Load, Eqn. #**
U = 1.4(D + F) + (**1.0**R0 + To) + 1.0C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + **1.6**Ro + 1.2To) + 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + **0.8**Ro + 1.2To) + 1.4C + 1.6(Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.0W + 1.0C + 0.5(Lr or S or R)	SEL, (NB 2-4)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.6Eo + 1.0C + 0.2(Lr or S or R)	SEL, (NB 2-5)
U = D + F + **1.0**L + H + To + Ro + Es + C	EEL, (NB2-6)
U = D + F + **1.0**L + H + To + Ro + Wt	EEL, (NB2-7)
U = D + F + **1.0**L + H + Ta + Ra + 1.4Pa + C	AL, (NB2-8)
U = D + F + **1.0**L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es	AL, (NB2-9)

*Footnotes and caveats applicable to this table are given in the Attachment to this report.

The bold type font in this table highlights the differences between the AISC N690-18 LRFD load combinations and the recommended LRFD load combinations.

For extreme load combination NB2-9, the load factor of 0.7 for Es in N690 is appropriate for steel and SC structures. N690-18 load combinations invoke the principle of action-companion action load combination approach. This approach considers that when a primary load in a load combination peaks the other companion loads would not reach their peak value at the same time. The principal loads in load combination NB2-9 are the accident and impact loads which are taken at their peak maximum values while Es is a companion load and the probability of simultaneous occurrences of the peak Es with the accident and impact loads is small. Even for those low probability cases when Es and accident loads would be concurrent, the time scales of

Es and the accident loads are different; the peak response to SSE ground motion is reached in seconds while the peak response to an accident like a LOCA takes longer to reach, which leads to even smaller probabilities of coincident peak responses. If 0.7Es is used in load combination NB9-9, then the absolute sum method should be used to combine the dynamic loads. If 1.0Es is used in load combination NB2-9, then the SRSS method may be used. Explanation for the other changes shown in bold, for the proposed guidance for DG load combinations, is presented in the Attachment to this report.

Allowable Strength Design (ASD)

The ASD load combination approach is comparable to the allowable stress method presented in N690-1994 (R2004), and thus a direct comparison is possible. Table 3 below presents the ASD load combinations in N690-18. The ASD load combination approach in N690-18 Section NB2.6 has some notable differences from the allowable stress method presented in N690-1994 (R2004). These differences are primarily due to advances in the development of the N690 specification over the years and the principle of action-companion action load combination approach.

The ASD approach is established using the same limit states as the LRFD approach and using the load and resistance factors of the LRFD method to set the factor of safety and load combinations for the ASD method.

Table 3 Allowable Strength Design (ASD) Load Combinations in AISC N690-18

AISC N690-18 Section NB2 ASD*		Load, Eqn. #
U = (D + F) + (L + H + Ro + To)	+ C	NL, (NB2-10)
U = (D + F) + (H + Ro + To)	+ C + (Lr or S or R)	NL, (NB2-11)
U = (D + F) + (0.75L + 0.75H + To)	+ C + 0.75(Lr or S or R)	NL, (NB2-12)
U = (D + F) + (0.75L + 0.75H + Ro + To) + 0.6W	+ C + 0.75(Lr or S or R)	SEL, (NB2-13)
U = (D + F) + (0.75L + 0.75H + Ro + To) + Eo	+ C + 0.75(Lr or S or R)	SEL, (NB2-14)
U = D + F + L + H + To + Ro + Es	+ C	EEL, (NB2-15)
U = D + F + L + H + To + Ro + Wt		EEL, (NB2-16)
U = D + F + L + H + Ta + Ra + Pa	+ C	AL, (NB2-17)
U = D + F + L + H + Ta + Ra + Pa + Yr + Yj +Ym + 0.7Es		AL, (NB2-18)

*Footnotes and caveats applicable to this table are given in the Attachment to this report.

A review of the ASD load combinations in N690-18 shows that the SEL, EEL and AL load combinations for the ASD method align well with those for the LRFD method in N690-18. However, the NL load combinations, NB2-10 through NB2-12 are not consistent with load combinations NB2-1 through NB2-3 (Table 1 above). Given the general AISC approach for the development of the LRFD and the subsequent development of the new ASD approach, the NL load combinations should include the same loads for the ASD and LRFD methods. In addition, the principal loads in the equivalent load combinations should be the same. NB2-1 is essentially a dead load combination, NB2-2 is essentially a live load combination and NB2-3 is essentially a live load combination but with the roof/snow/rain loads as the principal loads. For this reason, NB2-10 should be modified to remove L and H, NB2-11 should be modified to add L and NB2-12 should be modified to add Ro. In this manner it is possible to compare the LRFD load combinations proposed for the DG with those for the ASD in N690-18 and propose modifications to the load factors for consistency between the load combinations for both methods.

In addition to modifying the NL load combinations for the ASD method to have the same loads as those for the LRFD method, a few load factors for all ASD load combinations in the N690-18 also had to be modified to be consistent with the LRFD load combinations proposed in the previous section. Explanation for the changes developed for the proposed guidance for DG load combinations is presented in the Attachment to this report. The resulting ASD load combinations proposed for this DG are in Table 4.

Table 4 - Proposed ASD Load Combinations*

Proposed Guidance for DG ASD		**Load, Eqn. #**
U = (D + F) + (Ro + To)	+ C	NL, (NB2-10)
U = (D + F) + (**L** + H + Ro + To)	+ C + **0.75**(Lr or S or R)	NL, (NB2-11)
U = (D + F) + (0.75L + 0.75H + **0.75Ro** + To)	+ C + **1.0**(Lr or S or R)	NL, (NB2-12)
U = (D + F) + (**1.0**L + **1.0**H + Ro + To) + 0.6W	+ C + 0.75(Lr or S or R)	SEL, (NB2-13)
U = (D + F) + (**1.0**L + **1.0**H + Ro + To) + Eo	+ C + 0.75(Lr or S or R)	SEL, (NB2-14)
U = D + F + L + H + To + Ro + Es	+ C	EEL, (NB2-15)*
U = D + F + L + H + To + Ro + Wt		EEL, (NB2-16)*
U = D + F + L + H + Ta + Ra + Pa	+ C	AL, (NB2-17)*
U = D + F + L + H + Ta + Ra + Pa + Yr + Yj +Ym + 0.7Es		AL, (NB2-18)*

*Footnotes and caveats applicable to this table are given in the Attachment to this report.

4.3.3 NB3 Design Basis

Between N690-12 NB3 / AISC 360-10 B3 and N690-18 NB3 / AISC 360-16 B3, there was significant re-organization and revision of the content. Although there were several steps in the review process over time, in the interest of clarity, only the final content of NB3 and the referenced paragraphs of B3 are evaluated herein.

NB3 contains fourteen (14) subsections, as follows:

1. Design for Strength Using Load and Resistance Factor Design (LRFD) (incorporates B3.1 by reference)
2. Design for Strength Using Allowable Strength Design (ASD) (incorporates B3.2 by reference, plus minor addition)
3. Required Strength (complete replacement for B3.3)
4. Design of Connections and Supports (incorporates B3.4 by reference)
5. Design of Diaphragms and Collectors (incorporates B3.5 by reference)
6. Design of Anchorages to Concrete (incorporates B3.6 by reference)
7. Design for Stability (incorporates B3.7 by reference)
8. Design for Serviceability (incorporates B3.8 by reference, plus minor addition)
9. Design for Structural Integrity (incorporates B3.9 by reference)
10. Design for Ponding (incorporates B3.10 by reference)
11. Design for Fatigue (incorporates B3.11 by reference)
12. Design for Fire Conditions (incorporates B3.12 by reference)
13. Design for Corrosion Effects (incorporates B3.13 by reference)
14. Design Based on Ductility and Local Effects (unique to NB3, no counterpart in B3)

Except for NB3.14, which is unique, and NB3.3, which replaces B3.3, the evaluation of NB3 required evaluation of B3 in its entirety.

- AISC 360-16 subsections B3.4, B3.5, B3.6, B3.7, B3.8, B3.10, B3.11, and B3.12 all refer to other chapters or appendices in AISC 360-16, which are evaluated elsewhere in this report.

- B3.13, Design for Corrosion Effects, simply states: “Where corrosion could impair the strength or serviceability of a structure, structural components shall be designed to tolerate corrosion or shall be protected against corrosion.”

- Therefore, this evaluation concentrated on the specific content of NB3.1/B3.1; NB3.2/B3.2; NB3.3; NB3.9/B3.9; and NB3.14.

<u>NB3.1/B3.1 Design for Strength Using Load and Resistance Factor Design (LRFD)</u>

> “Design according to the provisions for LRFD satisfies the requirements of this Specification when the design strength of each structural component equals or exceeds the required strength determined on the basis of the LRFD load combinations. All provisions of this Specification, except for those in Section B3.2, shall apply.
>
> Design shall be performed in accordance with Equation B3-1: $R_u \leq \Phi R_n$ (B3-1), where R_u = required strength using LRFD load combinations; R_n = nominal strength; Φ = resistance factor; ΦR_n = design strength. The nominal strength, R_n, and the resistance factor, Φ, for the applicable limit states are specified in Chapters D through K.“
>
> This subsection re-iterates the design process for LRFD, which is considered to be acceptable.

<u>NB3.2/B3.2 Design for Strength Using Allowable Strength Design (ASD)</u>

> “Design according to the provisions for allowable strength design (ASD) satisfies the requirements of this Specification when the allowable strength of each structural component equals or exceeds the required strength determined on the basis of the ASD load combinations. All provisions of this Specification, except those of Section B3.1, shall apply.
>
> Design shall be performed in accordance with Equation B3-2: $R_a \leq R_n / \Omega$ (B3-2) where R_a = required strength using ASD load combinations; R_n = nominal strength; Ω = safety factor; R_n/Ω = allowable strength. The nominal strength, R_n, and the safety factor, Ω, for the applicable limit states are specified in Chapters D through K.
>
> It is permitted to multiply the allowable strength by the coefficients stipulated in Section NB2.6d(8). [added by NB3.2]”

This subsection re-iterates the design process for ASD, which is considered to be acceptable.

NB3.3 Required Strength

> "The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations stipulated in Section NB2.
>
> Design by elastic, inelastic or plastic analysis is permitted. Provisions for inelastic and plastic analysis are as stipulated in Appendix N1, Section N1.3, Design by Inelastic Analysis.
>
> The yield stress, modulus of elasticity, and proportional limit of steel shall be investigated and reduced, as appropriate, for temperatures in excess of 250°F (120°C).
>
> [User Note: Values for the reduction in material properties of structural steels exposed to elevated temperatures can be found in resources such as the Structural Alloys Handbook, published by Battelle, Columbus, OH, and in the ASME Boiler and Pressure Vessel Code, Section II, Part D, Material Properties. Sustained temperature above 700°F (370°C) may subject the material to creep rupture effects that need to be considered in the design. Properties for fire conditions of commonly used structural steels are tabulated in Specification Appendix 4, Table A-4.2.1.]"

BNL has recommended that certain limits be placed on the use of inelastic analysis methods for safety related steel structures in commercial nuclear power plants, consistent with current staff positions. This is discussed in Section 4.16 of this report.

NB3.9/B3.9 Design for Structural Integrity

> "When design for structural integrity is required by the applicable building code, the requirements in this section shall be met.
>
> (a) Column splices shall have a nominal tensile strength equal to or greater than D + L for the area tributary to the column between the splice and the splice or base immediately below, where D = nominal dead load, kips (N) and L = nominal live load, kips (N).
>
> (b) Beam and girder end connections shall have a minimum nominal axial tensile strength equal to (i) two-thirds of the required vertical shear strength for design according to Section B3.1 (LRFD) or (ii) the required vertical shear strength for design according to Section B3.2 (ASD), but not less than 10 kips in either case.
>
> (c) End connections of members bracing columns shall have a nominal tensile strength equal to or greater than (i) 1% of two-thirds of the required column axial strength at that level for design according to Section B3.1 (LRFD) or (ii) 1% of the required column axial strength at that level for design according to Section B3.2 (ASD).
>
> The strength requirements for structural integrity in this section shall be evaluated independently of other strength requirements. For the purpose of satisfying these requirements, bearing bolts in connections with short-slotted holes parallel to the direction of the tension force and inelastic deformation of the connection are permitted."

The AISC 360-16 Commentary associated with NB3.9/B3.9 states: "Section 1615 of the International Building Code (ICC, 2015) assigns structural integrity requirements to high-rise buildings in risk category III or IV, which means that the number of buildings to which this requirement currently applies is limited." Therefore, BNL concluded that NB3.9/B3.9 is <u>not applicable</u> to safety-related steel structures.

<u>NB3.14 Design Based on Ductility and Local Effects</u>

This subsection describes acceptable applications of inelastic analysis methods for impulsive and impactive loads. The evaluation of NB3.14 is included in Section 4.16 of this report. BNL concluded it is acceptable.

<u>Recommendation</u>

Except for NB3.9/B3.9, which is not applicable, the text of N690-18 NB3 and the incorporated-by-reference text of AISC 360-16 B3 are acceptable. This is <u>not</u> a blanket acceptance of the AISC 360-16 chapters that are referenced from within NB3/B3. BNL's evaluations of the referenced chapters are covered throughout this report.

4.3.4 NB4 Member Properties

It is noted that there is no Section NB4 listed in the N690-18 Table of Contents. AISC 360-16 Section B4 is "Member Properties" AISC 360-16 Section B4 is assumed to be incorporated in its entirety in N690-18.

It is difficult to correlate AISC 360-16 Section B4 with a specific section in N690-1994 (R2004). There are similarities with the provisions in Section Q1.9 Width-Thickness Ratios. Q1.9 references Table Q12 in Appendix QA. Table Q12 has similarities with Tables B4.1a and b, but due to the evolution of criteria and reformatting it is difficult to assess the differences. N690-1994 (R2004) subsections Q1.14.1, 1.14.2.1 and 1.14.4 are included in AISC 360-16 subsection B4.3.

The technical basis for AISC 360-16 Section B4 is discussed at length in the Commentary. AISC 360-16 Section B4 has evolved from the development of the LRFD specification and has its roots in earlier documents. It is noted that AISC 360-05 replaced LRFD 1999 and the 1989 ASD Specification. LRFD 1999 replaced LRFD 1993. Review of these older references may shed light on any differences with N690-1994 (R2004); however, it was concluded that a more thorough review was not warranted. AISC 360-16 Section B4 has evolved over a considerable period of time, and is based on referenced research, analysis, and test data.

4.3.5 NB5 Fabrication and Erection

Chapter NB5, Fabrication and Erection replaces B5 of AISC 360 with the following: "Shop drawings, fabrication, shop painting, erection and quality control shall meet the requirements in Chapter NM, Fabrication and Erection." See assessment of Chapter NM in this report.

4.3.6 NB6 Quality Control and Quality Assurance

Chapter NB6, Quality Control and Quality Assurance, replaces B6 of AISC 360 with the following: "Quality control and quality assurance activities shall satisfy the requirements stipulated in Section NA5, Quality Assurance, and Chapter NN, Quality Control and Quality Assurance." See assessment of Section NA5 and Chapter NN in this report.

4.3.7 NB7 Evaluation of Existing Structures

Chapter NB7, Evaluation of Existing Structures, replaces B7 of AISC 360 with the following: "Provisions for the evaluation of existing structures are presented in Appendix N5, Evaluation of Existing Structures." See assessment of Appendix N5 in this report.

4.4 Chapter NC Design for Stability

4.4.1 N690-18 and AISC 360-16 Comparison to N690-1994 (R2004)

Chapter NC includes all of Chapter C of AISC 360, plus 1 minor addition which adds the following item to the list of five in the first paragraph of C1: "(6) and the effects of elevated temperatures."

Chapter C of AISC 360-2016 also references Appendices 6, 7, and 8. N690-2018 Appendices N6, N7, and N8 reference AISC 360-2016 Appendices 6, 7, and 8 in their entirety.

The evaluation of the guidance in Chapter NC "Design for Stability" required evaluation of the contents of AISC 360-2016 Chapter C, Appendices 6, 7, and 8, and the corresponding Commentary sections in AISC 360-2016. The Commentary sections are lengthy and provide significant additional technical details.

In AISC 360-2016, there is an apparent distinction made between "stability" and "buckling". Only stability is discussed in Chapter C. However, the referenced Appendices 6, 7, and 8 discuss buckling considerations, and buckling considerations are discussed in Chapters E through I and K of AISC 360-16, which are referenced in their entirety by N690-18 Chapters NE through NI and NK.

AISC 360-16 contains the following definitions in the Glossary:

- *Buckling*†. Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.
- *Buckling strength*. Strength for instability limit states.
- *In-plane instability*†. Limit state involving buckling in the plane of the frame or the member.
- *Instability*†. Limit state reached in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry produces large displacement.
- *Stability*. Condition in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry does not produce large displacements.
- *Limit state*†. Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength limit state).

AISC 360-16 Chapter C, Section C3. CALCULATION OF AVAILABLE STRENGTH, states:

"For the direct analysis method of design, the available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D, E, F, G, H, I, J and K, as applicable, with no further consideration of overall structure stability. The effective length factor, K, of all members shall be taken as unity unless a smaller value can be justified by rational analysis."

N690-1994 (R2004) is organized and formatted differently from AISC 360-16. There is considerable discussion of "buckling" throughout the document, but limited discussion of "stability". AISC 360-16 Chapters E through I and K have been compared, to the maximum practical extent, to the corresponding content of N690-1994 (R2004). This comparison included consideration of buckling criteria, as applicable.

The information in AISC 360-16 related to "stability" and added to N690-18 by reference, greatly expands on the information in N690-1994 (R2004). BNL's evaluation concentrated on the reasonableness of the methodology specified in AISC 360-16 Chapter C to address "stability" of safety-related steel structures in nuclear facilities.

Several optional approaches are included in AISC 360-16, without clear guidance on the limitations. The direct analysis method for evaluating stability, discussed in AISC 360-16 Chapter C, explicitly includes all potentially important nonlinear effects in the computer analysis, including construction tolerances, component fabrication tolerances, other geometric imperfections, material nonlinearity, P-delta effects – overall and between floors, and stiffness reduction. The Commentary warns that many software packages will be unsuitable for such analysis, and provides two relatively simple benchmark problems, for verification. The Commentary also recommends that additional, more realistic benchmark problems be taken from the referenced studies. This is acceptable, provided adequate benchmarking exists for the software used.

The direct analysis method, when properly implemented, is considered acceptable. However, the acceptability of the alternate methods is contingent on satisfying the stated restrictions, which may be difficult to evaluate in practice.

The Commentary to N690-18, Chapter NC, states the following:

Add the following new paragraph to Section C1:

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

This is appropriate and acceptable, recognizing that safety-related steel structures in nuclear facilities may be subjected to elevated temperatures during normal, abnormal, and/or accident conditions.

There are two acceptance criteria that must be considered: (1) adequate resistance against collapse under compressive loading (strength limit state); and (2) limit on displacement such that the intended function of safety-related structures, systems, and components (SSCs) is not jeopardized (serviceability limit state).

Both "safety-related" steel structures and "important-to-safety" steel structures need to be designed for stability. Safety-related steel structures perform a direct safety function; e.g., support for or protection of safety-related SSCs. Important-to-safety steel structures are those whose collapse or excessive displacement would negatively impact the intended function of safety-related SSCs, even though the steel structure itself performs no safety-related function. This distinction may be significant when defining the performance goal for a specific steel structure.

4.4.2 Evolution of Guidance on Design for Stability in AISC Specifications

AISC has a standing committee, the Structural Stability Research Council (SSRC), that shapes the content of AISC guidance on Design for Stability. There have been several recent articles, presentations and video short courses covering this subject. AISC also published Steel Design Guide 28 in 2013 covering this topic. BNL reviewed the references listed below, to gain a better understanding of the evolution of the current guidance and its appropriate implementation:

(1) "Design for Stability", J. Liu, Purdue Presentation, Spring 2014, CE470DAM_S14.
(2) "Stability Analysis: It's not as Hard as You Think", C. M. Hewitt, September 2008, Modern Steel Construction, pp. 44-46.
(3) "The Evolution of Stability Provisions in the AISC Specification", C. J. Carter, Chief Structural Engineer – AISC, Steel Day Eve 2013.
(4) "A New Approach to Design for Stability", R. S. Nair, Teng & Associates, Inc., AISC Webinar, January 29, 2016.
(5) "Design for Stability using the 2010 AISC Specification", L. F. Geschwindner, Professor Emeritus – Penn State University, AISC Webinar, October 24, 2016.
(6) Steel Design Guide 28 - Stability Design of Steel Buildings (AISC, 2013)

The consistent message is that the Direct Analysis Method is the new norm, and that the Equivalent Length Method, which has been used for many years, has limitations on its applicability and has considerable uncertainty in developing the appropriate value of K for practical structural steel frame configurations.

Within the Direct Analysis Method, there are options to perform either a first-order analysis with utilization of B_1 and B_2 factors to simulate the second-order effects (Appendix 8), or to directly perform a second-order analysis. In either case, there is guidance for consideration of all effects that may contribute to the loss of structural stability, as follows:

1. Flexural, shear, and axial deformations – these are the member deformations and all other component and connection deformations that contribute to displacements of the structure;
2. Second-order effects – these are the increases that occur in forces and moments due to displacements of the structure induced by the loads, including both $P - \Delta$ effects (displacements of points of intersection of members) and $P – \delta$ effects (deformations of the members between points of intersection);
3. Geometric imperfections – these are the initial out-of-plumbness of the structure and the initial out-of-straightness of the members;

4. Stiffness reductions due to inelasticity – these are the effects of residual stresses; and,
5. Variability in component and system stiffness – these are the effects of variations in material and cross-sectional properties of members, as well as the other effects generally accounted for in the resistance factors (LRFD) and safety factors (ASD).

Steel Design Guide 28 - Stability Design of Steel Buildings, states:

1.1 PURPOSE OF THIS DESIGN GUIDE

With the 2005 AISC Specification for Structural Steel Buildings (AISC, 2005a), hereafter referred to as the AISC Specification, the state of the art was advanced to include three methods for stability design, including the introduction of a powerful new approach - the direct analysis method (DM). The DM is a practical alternative to the more traditional effective length method (ELM), which has been the basis of stability considerations in earlier editions of the AISC Specification and continues to be permitted. In addition, the third method provided is a streamlined design procedure called the first-order analysis method (FOM), which is based upon the DM with a number of conservative simplifications.

The primary purpose of this Design Guide is to discuss the application of each of the aforementioned three methods and to introduce the DM to practicing engineers………. As explained in Chapter C and in this Design Guide, the DM is required in cases where the second-order effects due to sidesway are significant.

The Equivalent Length Method, which formerly was in Chapter C, has been relocated to an appendix. At the same time, the Direct Method, which formerly was in an appendix, has been relocated to Chapter C.

Some of the attractive features of the DM include:

- There is no need to calculate K factors.
- The internal forces are represented more accurately at the ultimate limit state.
- The method applies in a logical and consistent manner for all types of steel frames, including braced frames, moment frames, and combined framing systems.

Other purposes of this Design Guide are as follows:

- ………
- Describe the traditional ELM and update designers on new conditions placed on its use.

C.J. Carter, 2013, states:

Excerpt from “Developments Leading to the 1963 AISC Specification”:

“Additionally, the introduction of K was not without compromise. The methods available to calculate K in all but the most simple of cases required a number of assumptions, few and often none of which were actually satisfied in real structures (Kavanaugh, 1962). Regardless, what would come to be known as the effective length method was accepted because it did something and that was better than doing nothing."

4.4.3 Recommendations

After detailed review of AISC Specification 360-16, Chapter C – Design for Stability (and Appendices 6, 7, and 8); the associated Commentary; and the references cited above, BNL recommends acceptance of Chapter C, subject to the following qualifications:

(1) the provisions of Appendix 7 for the Equivalent Length Method (ELM) should ONLY be implemented for safety-related steel structures in nuclear facilities in cases where any restrictions on its use are clearly satisfied AND minimal judgement is required to determine K;

(2) the First Order Analysis Method (FOM), using B_1 and B_2 factors from Appendix 8 to simulate second order effects, should ONLY be implemented for safety-related steel structures in nuclear facilities in cases where any restrictions on its use are clearly satisfied;

(3) Prediction of elastic stability using the Direct Second Order Analysis Method (DM) is acceptable. Since it only considers geometric nonlinearities and initial imperfections, the stress state at the onset of instability should be confirmed to be in the elastic range to ensure valid results.

    The DM, considering both geometric and material nonlinearities, and accounting for initial imperfections, will provide the most accurate predictions of structural collapse due to instability or excessive deformation. When accurate displacement predictions are needed to satisfy a maximum displacement criterion that exceeds the elastic response limit (e.g., to preclude structure-to-structure interaction), this is the only viable analysis method.

4.5 Chapter ND Design of Members for Tension

4.5.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter ND includes all of Chapter D of AISC 360 with no changes.

D1, Slenderness Limitations, is similar to Section Q1.8.4 of N690-1994 (R2004).

D2, Tensile Strength, Eq D2-1 and -2 are the same as in Section Q1.5.1.1. The treatment of slot welds is the same as Q1.14.2.1.

D3, Effective Net Area, is similar to Q1.14.2.1 and Q1.14.2.2 with some new items. Table D3.1 has similar shear lag factors as Q1.14.2.2 but has additional items. Table D3.1 does not address U, the shear lag factor, for two fasteners as in Q1.14.2.2.

D4, Built-Up Members, refers to J3.5 for the maximum spacing of connectors. J3.5 is different when compared to Q1.18.3.1 (N690-1994 (R2004)). J3.5 spacing is 12 times "t" compared to 14 to 24 times “t” in Q1.18.3.1. Based on the Commentary for J3.5 (Pg. 402), D4 appears to be more conservative. The discussion on cover plates is the same as Q1.18.3.2 and the User Note is similar to Q1.18.3.2.

The provisions in D5, Pin-Connected Members, have been updated compared to the provisions in Q1.5.1.1 and Q1.14.5. Some similarities exist with regard to dimensional requirements. Further review regarding the basis for the changes would be needed in order to assess the differences.

The provisions in D6, Eyebars, appear to be the same as Q1.14.5. D6.1 refers to D2 for tensile strength. Q1.14.5 does not specifically refer to Q1.5.1.1 for tensile strength for eyebars. However, it seems that this section is meant to be applied.

As noted above, there are some differences compared to N690-1994 (R2004). Further review was not considered to be warranted, in order to track down the basis for the updated requirements.

4.5.2 N690-18 Comparison to N690-2012

No changes to Chapter ND.

4.5.3 AISC 360-16 Comparison to AISC 360-10

AISC 360-16 added a shear lag factor for welded plates or connected elements with unequal length longitudinal welds.

Section D3 of the Commentary explains the change as follows:

> Prior to 2016, two plates connected with welds shorter in length than the distance between the welds were not accommodated in Table D3.1. In light of the need for this condition, a shear lag factor was derived and is now shown in Case 4. The shear lag factor is based on a fixed-fixed beam model for the welded section of the connected part. The derivation of the factor is presented in Fortney and Thornton (2012).

Sufficient justification is provided for this change.

There were some other modifications and updating to the Specification and Commentary; however, based on a cursory review, a more detailed review is not warranted.

4.6 Chapter NE Design of Members for Compression

4.6.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter NE includes all of Chapter E of AISC 360 with no changes.

E1, General Provisions, states that compressive strength is the lowest limit state of flexural buckling, torsional buckling, and flexural-torsional buckling. Q1.5.1.3 of N690-1994 (R2004) addresses the same items.

E2, Effective Length, refers to KL/r as does Q.1.8.1. The User Note referring to the KL/r limit of 200 is the same as Q1.8.4, line 1.

E3, Flexural Buckling of Members Without Slender Limits, is difficult to compare since the basis for the equations in this section has changed. Eq. E3-3 is close to Eq. Q1.5-2 in Q1.5.1.3.2. The Commentary of the 1999 LRFD Specification states that Eq E3-2 and 3 are based on a reasonable conversion of research data into design equations. The conversion of the ASD equation, which was based on the CRC (Column Research Council) curve was found to be cumbersome. The new equations are essentially the same as column-strength curve 2P of the Structural Stability Research Council. Efforts were made to equate ASD and LRFD expressions at a slenderness ratio of 1.5. Per AISC 2005 Commentary on E3, for both LRFD and ASD, the new column equations give somewhat more economy than previous editions of the Specification. Commentary Fig C-E3.2 (pg 293) compares ASD column curves for 2005 and 1989. At KL/r < 50, new curve provides slightly higher allowable stress. Fig. C-E3.1 shows differences for LRFD curves for 2005 and 1999. These differences (say less than 5%) are larger than for ASD curves. They also extend across all Kl/r values.

E4, Torsional and Flexural-Torsional Buckling of Members Without Slender Elements, is addressed in a very general way in Q1.5.1.3.6. The 1994 Commentary refers to App E3 of 1985 LRFD Spec for critical elastic buckling stress (reference not available). However, the equations in E4 evolve at least from the 1999 LRFD Specification. The Commentary for E4 and the Commentary for Appendix E of the 1999 LRFD Specification cite the same references. The equations in E4 appear in the 1999 LRFD Specification, Section E and Appendix E.

E5, Single Angle Compression Members is not addressed in N690-1994 (R2004). The equations in E5 are the same as in AISC 360-2005, but do not appear in the1999 LRFD Spec. The criteria appear to have evolved over time.

E6, Built-up Members, are addressed in Q1.18 of N690-1994, but coverage is different and cannot be correlated to the information in E6. Q1.5.1.3, Compression, does not cover built-up members. Information in E6 has evolved from the 1999 LRFD Specification. Eq E6-1 is the same as Eq E4-1 in E4 of the 1999 Specification; however, Eq E6-2a and 2b are different from Eq E4-1. Information on Dimensional Requirements is similar to the 1999 Specification with additions.

E7, Members with Slender Elements, is not covered in a similar manner in N690-1994 (R2004). The Commentary for E7 states that when designing with hot-rolled shapes, the engineer will seldom find occasion to use E7. The Commentary indicates that the approach in this section has been used since the 1969 AISC Specification. Equations in E7.1 and E7.2 are also in the 1999 LRFD Specification, Appendix B5.3.

As noted above, there are differences compared to N690-1994 (R2004). Comparison between the older and new specification are difficult. The newer specification also has much more information, much of which can be traced back to the 1999 LRFD Specification or other earlier standards. It was concluded that further review to track down the basis for the updated requirements was not warranted.

4.6.2 N690-18 Comparison to N690-2012

No changes to Chapter NE.

4.6.3 AISC 360-16 Comparison to AISC 360-10

AISC 360-16 indicates that the available compressive strength for double angles and tees is determined by the general flexural-torsional buckling equation for members without slender elements.

Section E4 of the Commentary explains the change as follows:

> The specific method of calculating the buckling strength of double-angle and tee-shaped members that had been given in the 2010 AISC *Specification* (AISC, 2010) has been deleted in preference for the use of the general flexural-torsional buckling equations because the deleted equation was usually more conservative than necessary.

This change resulted in the deletion of Equation E4-2 in AISC 360-10 and refers back to Equations E3-2 or E3-3. This justification appears to be reasonable and further review does not appear to be warranted.

AISC 360-16 added a constrained-axis torsional buckling limit state for members with lateral bracing offset from the shear center.

Section E4(d) of the Specification states:

> For members with lateral bracing offset from the shear center, the elastic buckling stress, *Fe*, shall be determined by analysis.
>
> **User Note:** Members with lateral bracing offset from the shear center are susceptible to constrained-axis torsional buckling, which is discussed in the Commentary.

Section E4 of the Commentary states:

> Many common bracing details may result in situations where the lateral bracing is offset from the shear center of the section, such as columns or roof trusses restrained by a shear diaphragm that is connected to girts or purlins on the outside of the column or chord flange. Depending on the orientation of the primary member, the bracing may be offset along either the minor axis or the major axis as depicted in Figure C-E4.2. Since girts or purlins often have relatively simple connections that do not restrain twist, columns or truss chords can be susceptible to torsional buckling. However, in common cases due to the offset of the bracing relative to the shear center, the members are susceptible to constrained-axis torsional buckling. Timoshenko and Gere (1961) developed the following expressions for constrained-axis torsional buckling:

The Commentary goes on to discuss the equations by Timoshenko and Gere.

Sufficient justification has been provided for this change.

AISC 360-16 revised the available compressive strength formulation for members with slender compression elements.

Section E7 of the Commentary provides an extensive discussion of members with slender elements. The Commentary includes the following discussion on the evolution of the Specification on this topic:

> The *Q*-factor approach to dealing with columns with slender elements was adopted in the 1969 AISC *Specification* (AISC, 1969), emulating the 1969 AISI *Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 1969). Prior to 1969, the AISC practice was to remove the width of the plate that exceeded the λ_r limit and check the remaining cross section for conformance with the allowable stress, which proved inefficient and uneconomical. Two separate philosophies were used: Unstiffened elements were considered to have attained their limit state when they reach the theoretical local buckling stress; stiffened elements, on the other hand, make use of the post-buckling strength inherent in a plate that is supported on both of its longitudinal edges, such as in HSS columns and webs of I-shaped columns. The effective width concept is used to obtain the added post-buckling strength. This dual philosophy reflects the 1969 practice in the design of cold-formed columns. Subsequent editions of the AISI Specifications, in particular, the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001, 2007, 2012), hereafter referred to as the AISI *North American Specification,* adopted the effective width concept for both stiffened and unstiffened elements. This approach is adopted in this Specification.

The Commentary further states:

> The impact of the changes in this Specification for treatment of slender element compression members is greatest for unstiffened element compression members and may be negligible for stiffened element compression members as shown by Geschwindner and Troemner (2016).

The title of the above reference is:

> Geschwindner, L.F. and Troemner, M. (2016), “Notes on the AISC 360-16 Provisions for Slender Compression Elements in Compression Members,” *Engineering Journal*, AISC, Vol. 53, No. 3, pp. 137–146.

The Commentary also provides a number of other references on this subject.

Sufficient justification is provided for this change.

Other changes to Chapter E are discussed below.

In E1 of Commentary the following paragraph replaced the end of the second paragraph and the third paragraph:

> Since then, a number of changes in industry practice have taken place: (a) welded built-up shapes are no longer manufactured from universal mill plates; (b) the most commonly used structural steel is now ASTM A992/A992M, with a specified minimum yield stress of 50 ksi (345 MPa); and (c) changes in steelmaking practice have resulted in materials of higher quality and much better defined properties. The level and variability of the yield stress thus have led to a reduced coefficient of variation for the relevant material properties (Bartlett et al., 2003).

There were various changes to Section E4 and Section E7 as discussed above. There were other changes noted that do not warrant further review.

Section E2, Effective Length, and subsequent equations were revised as discussed in the Commentary below:

> In the 2016 AISC *Specification*, the effective length, which since the 1963 AISC *Specification* (AISC, 1963) had been given as *KL*, is changed to *Lc*. This was done to simplify the definition of effective length for the various modes of buckling without having to define a specific effective length factor, *K*. The effective length is then defined as *KL* in those situations where effective length factors, *K*, are appropriate. This change recognizes that there are several ways to determine the effective length that do not involve the direct determination of an effective length factor. It also recognizes that for some modes of buckling, such as torsional and flexural-torsional buckling, the traditional use of *K* is not the best approach. The direct use of effective length without the *K*-factor can be seen as a return to the approach used in the 1961 AISC *Specification* (AISC, 1961), when column strength equations based on effective length were first introduced by AISC.

This change is acceptable and involves various changes to the equations that involve effective length as discussed above in the Commentary.

4.7 Chapter NF Design of Members for Flexure

4.7.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter NF includes all of Chapter F of AISC 360 with no changes.

F1 is general provisions. It specifies Eq F1-1 for the lateral-torsional buckling modification factor (Cb). This equation is also in the 1999 LRFD Specification. N690-1994 (R2004) provides a different equation in Q1.5.1.4.5 (pg 16) for the same factor. The AISC 360-10 Commentary (pg. 16.1-304) provides a suitable explanation for the change.

F2 provides the nominal flexural strength for the limit state of yielding in Eq F2-1 (Mn=Mp=FyZx). This section covers doubly symmetric I-shaped members and channels. The corresponding equation in N690-1994 (R2004) is in Q1.5.1.4.1 and is given as Fb=0.66Fy. The increase factor of 1.1 above the normal allowable of 0.6Fy apparently is in recognition of the lower bound shape factor (Z/S) for such compact sections. Since F2 is based on Z, the increase factor varies depending on the member used. Based on section property data from AISC, the shape factors were calculated and can vary (in percentage) for various shapes as follows: W:1.1-1.3, M:1.1-1.19, S:1.15-1.21, HP:1.1-1.15, C:1.16-1.29, MC:1.17-1.29. Thus, depending upon the section used, AISC 360-10 would allow up to an 18% (1.3/1.1) increase in stress above the N690-1994 allowable.

It is stated in the F1 Commentary, that for most designs, the engineer need seldom go beyond Section F2. It is also stated in the F2 Commentary that the equations in F2 are identical to the corresponding equations in F1 of the 1999 LRFD Specification. However, it is noted that the stress at the interface between inelastic and elastic buckling has been changed from Fy – Fr to 0.7Fy. It is also stated that the AISC 360-10 provisions have been simplified when compared to the previous ASD provisions based on a more informed understanding of beam limit state behavior and that the maximum allowable stress may be slightly higher than the previous limit of 0.66Fy.

For the lateral-torsional buckling limit state, Eq F2-2 is like the 1999 LRFD Specification except for the change to use 0.7Fy. Q1.5.1.4.5 addresses this subject and required three equations to be checked. The changes are adequately addressed in the F2 Commentary.

F3 to F5 address less commonly used members. Q1.5.1.4.2 covers the transition between compact and noncompact flanges. Further study would be needed to assess the significance of the differences between the old and new provisions.

F6 covers I-shaped members and channels bent about their minor axis. For the limit state of yielding, Eq F6-1 states Mn=Mp=FyZy<1.6FySy. Thus, the increase must be less than or equal to a shape factor of 1.6. Based on section property data from AISC, the shape factors were calculated and can vary (in percentage) for various shapes as follows: W:1.51-1.63, M:1.53-1.62, S:1.71-1.86, HP:1.53-1.56, C:2.00-2.29, MC:1.86-2.14. For such sections, N690-1994 (R2004), Section Q1.5.1.4.3 specifies Fb=0.75Fy, which is equivalent to an increase of 1.25. Thus, depending upon the section used, AISC 360-10 would allow up to a 28% (1.6/1.25) increase in stress above the N690-1994 allowable. The commentary in CQ1.5.1.4.3 indicates that although the shape factor for these members bent in this direction is considerably more than 1.25, full advantage is not taken in order to provide elastic behavior at service loading. The AISC 360-10 and 2005 Commentaries do not discuss this and the 1999 LRFD Specification did not address this loading condition. The current code at least acknowledges this concern by the limit of 1.6.

F7 and F8 cover square and rectangular HSS and box shaped members and round HSS. As in the other sections above, use is made of the plastic section modulus, which would increase the allowable stresses above previous ASD provisions. Q1.5.1.4.4 in N690-1994 (R2004) covers box members and specify allowables of 0.6Fy to 0.66Fy.

F9 through F13 cover other members. It is not clear how these members were addressed in N690-1994 (R2004). It is noted that some of these sections appear to have been revised from the 2005 specification.

As stated in the AISC 360-10 Commentary, for most designs, the engineer need seldom go beyond Section F2. Therefore, it was concluded that further review of the other sections was not warranted. The most significant change appears to be the use of the plastic section modulus in the allowable strength equations. Depending upon the section used, AISC 360-10 would allow up to an 18% (1.3/1.1) increase in stress above the N690-1994 allowable. As noted in the review, the increase in allowable stress would be even greater for members bent about their minor axis. The use of the plastic section modulus in the ASD method is considered to be acceptable since the approach in N690-18 was established so that the design strength of members are comparable when using the LRFD method or the ASD method.

4.7.2 N690-18 Comparison to N690-2012

No changes to Chapter NF.

4.7.3 AISC 360-16 Comparison to AISC 360-10

AISC reformulated the available flexural strength provisions for tees and double angles.

Section F9 of the Commentary provides the following explanation on this subject:

> This section addresses both tees and double angles loaded in the plane of symmetry. Prior editions of the Specification did not distinguish between tees and double angles and as a result, there were instances when double angles would appear to have less strength than two single angles. This Specification has addressed this concern by providing separate provisions for tees and double angles. In those cases where double angles should have the same strength as two single angles, the provisions reference Section F10.

The Commentary provided an extensive discussion of the basis for the provisions in the section.

Sufficient justification is provided for this change.

There were various other changes to the Specification and Commentary. The most significant change appears to be to Section F9 which is discussed above. Further review of the other changes is not warranted.

4.8 Chapter NG Design of Members for Shear

4.8.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter NG includes all of Chapter G of AISC 360 with no changes.

In AISC 360-10, the nominal shear strength of a web is defined by Eq G2-1 as the product of shear yield force (0.6FyAw) and the shear buckling reduction factor (Cv). For many rolled I-shapes, G2.1(a) defines Omega as 1.5 for ASD and Cv=1.0. This is the same as the shear allowable stress in Q1.5.1.2.1 of N690-1994 (R2004). For other doubly symmetric shapes and singly symmetric shapes and channels, except round HSS, G2.1(b) defines the web shear coefficient, Cv, to be used. The equations for Cv can be traced back to 1999 LRFD, Appendix F2. Also, Q1.10.5.2 of N690-1994 (R2004) provides similar equations for plate girders and rolled beams. The equation G2-6 for the shear buckling coefficient (Kv) is like 1999 LRFD Appendix F2 (pg 102, 110), but differs a little from the equations in Q1.10.5.2.

It is noted in the AISC 360-10 Commentary that Eq G2-6 (Eq C-G2-3 in the Commentary) applies as long as there are flanges on both edges of the web. The Commentary states that for tee-shaped beams, Kv=1.2. The Commentary also states that Section G2.1 assumes monotonically increasing loads. For members subjected to load reversals, such as earthquakes, special design considerations may apply (Popov, 1980). It would appear that such guidance should be in the Specification.

It is noted that Q1.5.1.2.2 addresses shear at beam end connections. These provisions are addressed in a similar manner in Section J4.3 Block Shear Strength of AISC 360-10.

Section G2.2, Transverse Stiffeners, is similar to the1999 LRFD, Appendix F2.3, but is difficult to compare to Q1.10.5.3 and Q1.10.5.4 which covers Intermediate Stiffener Spacing and Design. The AISC 360-10 Commentary states that the moment of inertia of the stiffener is the same as in AASHTO 2010, but is different from the 1989 AISC Allowable Stress Design. It was concluded that further review to assess any differences was not warranted.

Section G3, Tension Field Action, can be traced back to the 1999 LRFD, Appendix G3. Eq G3-2 is also the same as Eq Q1.10-2 in N690-1994 (R2004). Eq G3-4 replaces the AISC 360-05 transverse stiffener area requirement as explained in the AISC 360-10 Commentary (pg 328). This equation is the same as in AASHTO 2010.

It is not apparent as to how the requirements in G4 Single Angles, G5 Rectangular HSS and Box-Shaped Members, and G6 Round HSS are covered in N690-1994 (R2004). These provisions are almost the same as AISC 360-05. However, G4 of AISC 360-05 had Cv=1.0 and now reference is made to G2.1(b) with no explanation for the change in the Commentary. In G5, “t” is defined with no explanation in the Commentary. These do not appear to be significant changes.

G7 covers Weak Axis Shear in Doubly Symmetric and Singly Symmetric Shapes. This does not appear to be specifically addressed in N690-1994 (R2004). Provisions are similar to AISC 360-05, however, “b” is now defined along with a related clarification.

G8 Beams and Girders with Web Openings provides very general requirements. AISC 360-05 is the same. The Commentary provides information on one general procedure for assessing these effects. This subject does not appear to be addressed in N690-1994 (R2004).

As noted in the discussion above, there are differences compared to N690-1994 (R2004). Comparison between the older and new specification are difficult. The newer specification also has much more information, much of which can be traced back to the 1999 LRFD Specification or other earlier standards. There do not appear to be any significant differences in AISC 360-10 with the shear provisions in N690-1994 (R2004) (Q1.5.1.2 and Q1.10.5.2). It was concluded that further review to assess any significant differences with other provisions in N690-1994 was not warranted.

It is noted that some guidance in the AISC 360-10 Commentary related to Section G2.1 should be discussed in the Specification.

4.8.2 N690-18 Comparison to N690-2012

No changes to Chapter NG.

4.8.3 AISC 360-16 Comparison to AISC 360-10

AISC 360-16 revised the shear strength of webs of certain I-shapes and channels without tension field action and when considering tension field action.

The design of I-shaped members and channels is covered in Section G2 of the Specification. The Commentary provides an extensive discussion of this subject.
It is difficult to separate what appeared before and what are the current revisions. However, the provisions of this section appear to be well documented. Therefore, further review of the changes does not appear to be warranted.

The Chapter G Specification and Commentary were revised in a number of places. Some of the changes are characterized above, but go beyond Chapter G2. As discussed above, it is difficult to separate what appeared before and what are the current revisions. However, the provisions of this section appear to be well documented. Therefore, further review of the changes does not appear to be warranted.

4.9 Chapter NH Design of Members for Combined Forces and Torsion

4.9.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter NH includes all of Chapter H of AISC 360 with no changes.

Section H1 of AISC 360-10 addresses doubly and singly symmetric members subjected to flexure and axial force. The Commentary states that a new approach to the interaction of flexural and axial forces was introduced in the 1986 LRFD Specification and provides an explanation of the thinking behind the interaction curves used.

The Commentary to the 1986 LRFD Specification states that the equations for flexure and compression (Eq H1-1a and H1-1b in Section H1.1 of AISC 360-10) are simplifications and clarifications of similar equations used in the AISC Specification since 1961. The 1986 Commentary also states that in the development of these formulations, a number of alternative formulations were compared to the exact solutions of 82 sidesway cases reported in a 1977 University of Texas report by T. Kanchanalai. The 1986 Commentary goes on to explain that Eq H1-1a and H1-1b are similar in application to the previous Eq Q1.6-1a of the 1978 AISC Specification (also in N690-1994 (R2004)). When Cm=1, the factors 8/9 and 1/2 make the new equations more liberal than Q1.6-1a. For Cm<1 the new equations will be slightly more conservative. For the entire range of l/r and Cm, it is stated that the equations compare very closely to exact inelastic solutions of braced members.

The AISC 360-10 Commentary discusses Eq C-H1-1 which is similar to Eq Q1.6-1a of N690-1994 (R2004). It is difficult to compare Eq Q1.6-1a to the current Eq H1-1a, so based on the discussion in the 1986 LRFD Commentary, it is accepted that the current equations are simplifications and clarifications of the earlier AISC equations. The AISC 360-10 Commentary presents in Figure C-H1.1 for major-axis flexure, a comparison of Eq H1-1 with an “exact” solution for a W8x31. The two curves almost overlap each other. The “exact” solution is apparently based on equations in ASCE, 1971. Figure C-H1.2 also presents comparisons for minor-axis flexure, which for this direction, demonstrates that the AISC interaction curves are very conservative.

Section H1.2 addresses members subject to flexure and tension. It refers back to Eq H1-1a and H1-1b with appropriate references to Section D for the definition of the terms used in the equations. This subject is discussed in Q1.6.2 of N690-1994 (R2004). As in the case of compression above, the equations are different, but the issues appear to be handled in a consistent manner. As will be discussed further below, Eq Q1.6-3 in Section Q1.6.2 is basically the same as Eq H2-1 in Section H2 of AISC 360-10.

Section H1.3 covers doubly symmetric rolled compact members subject to single axis flexure and compression. The Commentary states that this section gives an optional equation for checking the out-of-plane resistance of certain types of beam-columns. As explained in the Commentary, the approach is changed from the 2005 Specification and states that the Eq H1-2 in this section allows a larger bending moment over most of the applicable range. The Commentary provides a reasonable discussion of the less conservative approach given in this section. N690-1994 (R2004) does not have a discussion of this subject.

Section H2 addresses the interaction of flexure and axial stress for shapes not covered in Section H1. It is permitted to use Section H2 for any shape in lieu of the provisions of Section

H1. The AISC 360-10 Commentary states that the equations in Section H1 are more liberal than those in Section H2. The Commentary also notes that the designer will have to consult the provisions of Sections H2 (and H3) only in rarely occurring cases. Equation H2-1 in Section H2 is essentially the same as Equation Q1.6-2 in Section Q1.6.1 of N690-1994 (R2004) and Equation Q1.6-3 in Section Q1.6.2. This would imply that the equations in H1 are more liberal than the equations in Section Q1.6.1 of N690-1994 (R2004).

Section H3 addresses members subject to torsion and combined torsion, flexure, shear and/or axial stress. As noted above, the Commentary states that the designer will have to consult the provisions of Section H3 only in rarely occurring cases. The first two parts address the design of HSS members, and the third part is a general provision that covers all other cases. Under the general provision “the required stresses are determined by elastic stress analysis based on established theories of structural mechanics. The AISC 360-10 Commentary refers the reader to AISC Design Guide 9, Torsional Analysis of Structural Steel Members (Seaburg and Carter, 1997). Section Q1.6.4 of N690-1994 (R2004) simply states that the effects of torsion shall be considered and the normal and shearing stresses due to torsion shall be added to those from all other loads, with resultants not exceeding the allowable values. The N690-1994 (R2004) Commentary simply references AISC Torsional Analysis of Steel Members (1983).

Section H4 addresses the rupture of flanges with holes subject to tension. Eq H4-1 is provided to evaluate limit states of tensile rupture of the flanges of beam-columns. The section was added after 2005 and the source of the provision is not discussed in the Commentary. The provision follows the same basic interaction guidelines that are used in the rest of Section H. A similar provision could not be found in N690-1994 (R2004).

In summary, the AISC 360-10 provisions provide simplifications and clarifications of similar equations used in allowable stress design since 1961. They also provide updated and expanded coverage of areas not previously addressed. As noted in the AISC 360-10 Commentary, some of the newer provisions are less conservative, than the previous specification. It was concluded that further review to track down the basis for the updated requirements was not warranted.

4.9.2 N690-18 Comparison to N690-2012

No changes to Chapter NH.

4.9.3 AISC 360-16 Comparison to AISC 360-10

There do not appear to be any significant changes to Chapter H. Further review of this Chapter does not appear to be warranted.

4.10 Chapter NI Design of Composite Members

4.10.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter NI includes all of Chapter I of AISC 360, with substitution of ACI-349 for ACI-318 in two places.

Chapter I of AISC 360-10 addresses composite members composed of rolled or built-up structural steel shapes or HSS and structural concrete acting together, and steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with steel headed stud anchors, concrete-encased, and concrete filled beams, constructed with or without temporary shores, are included. Chapter I addresses general provisions, axial force, flexure, shear, combined axial force and flexure, load transfer, composite diaphragms and collector beams, steel anchors, and special cases. “Steel anchor” replaces the old term “shear connector.” For special cases, when composite construction does not conform to the requirements of this section, the strength of steel anchors and details of construction shall be established by testing.

Section Q1.11 of N690-1994 (R2004) addresses composite construction. Composite construction is defined as steel beams or girders supporting a reinforced concrete slab, so interconnected that the beam and slab act together to resist bending. The subsections address encased beams, non-encased beams using shear connectors, end shear, shear connectors, and composite beams or girders with formed steel deck. For special cases, when composite construction does not conform to the requirements of Section Q1.11, allowable load per shear connector must be established by a suitable test program.

AISC 360-10 includes new material and significant changes compared to AISC 360-05. New sections covering shear, load transfer, composite diaphragms and collector beams, and steel anchors were added to AISC 360-10, as well many other changes summarized in the Commentary. AISC 360-05 already had extensive technical and format changes and new material compared to previous editions such as the 1999 and 1986 LRFD Specifications. The 1999 LRFD is like the 1986 LRFD with some additions and/or modifications and an expanded commentary. N690-1994 (R2004) had a different format compared to all the other specifications (1999, 2005 and 2010). It is difficult to correlate the new AISC 360-10 specification with the previous specifications.

During the review it was observed that the definition of a concrete-encased beam was simplified to a “beam totally encased in concrete cast integrally with slab.” This definition was included in AISC 360-10 and AISC 360-05. However, the 1986 and 1999 LRFD Specifications had the following definition: “A beam totally encased in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided that: (1) concrete cover over beam sides and soffit is at least 2 in.; (2) the top of the beam is at least 1 1/2 in. below the top and 2 in. above the bottom of the slab; and (3) concrete encasement contains adequate mesh or other reinforcing steel to prevent spalling of concrete.” Section Q1.11.1 of N690-1994 (R2004) had a similar definition.

There is some confusion about the use of steel anchors for encased composite members. Apparently, they are only required when using Method (c) of Section I3.3 of AISC 360-10. The AISC 360-10 Commentary supports this conclusion. However, I6.4a of AISC 360-10 seems to require steel anchors both within and outside the load introduction length. N690-1994 (R2004), as well as the 1986 and 1999 LRFD Specifications, did not require anchors if the restrictions imposed on encasement as described above are met. The AISC 360-10 Specification, as well AISC 360-05, no longer has the above restrictions imposed on encasement. If this is found to be confusing to the designer, the matter could be resolved by the AISC specification committee.

In summary, there are differences compared to N690-1994 (2004). Comparison between the older and new specification is difficult to achieve. AISC 360-10 includes new material and significant changes compared to AISC 360-05. AISC 360-05 already had extensive technical and format changes and new material compared to previous editions such as the 1999 and 1986 LRFD Specifications. It was concluded that further review to track down the basis for the updated requirements was not warranted.

4.10.2 N690-18 Comparison to N690-2012

N690-18 Chapter NI proposes to modify Chapter I of the Specification as follows:

Replace “ACI 318, Chapter 17” with “ACI 349, Appendix D” and Replace “ACI 318” with “ACI 349.”

Generally, throughout N690-18, ACI 349 should be identified rather than ACI 318, since ACI 349 is applicable to nuclear safety-related concrete structures. Furthermore, the NRC currently endorses ACI 349-13 with some regulatory positions in RG 1.142 Rev. 3. Therefore, it is recommended that when implementing N690-18 and the referenced AISC 360-16 specification, ACI 349-13 and RG 1.142 Rev. 3 should be followed, unless justified otherwise.

Evaluation of Commentary to N690-18

The following was included in N690-12 and N690-18 with different year references:

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

The same assessment for using ACI 349-13 with regulatory positions in RG 1.142 Rev. 3, discussed above would also be applicable here, unless otherwise justified.

4.10.3 AISC 360-16 Comparison to AISC 360-10

AISC 360-16 increased the limit on rebar strength to 80 ksi for composite columns.

Section I.3 of the Commentary states that:

> The specified minimum yield stress of reinforcing bars has been increased to 80 ksi (550 MPa) in coordination with ACI 318 (2014).

As discussed above, ACI 349-13 with regulatory positions in RG 1.142 Rev. 3 would be applicable here as well. However, RG 1.1.42 Rev. 3 does not endorse, in general, the use of HS (high strength) reinforcement (Grades 75 and 80) as used in ACI 349-13. Therefore, the use of HS reinforcement (greater than 60 ksi) should not be permitted, unless justified.

AISC 360-16 incorporated provisions for applying the direct analysis method to composite members. Section I1.5 of the Commentary, which addresses the stiffness for calculation of required strengths, states:

> This section along with Chapter C forms the basis of the direct analysis method of design for structural systems including encased composite members or filled composite members…
>
> Research indicates that the stiffness prescribed in this section may result in unconservative errors for very stability sensitive structures (Denavit et al., 2016a).

> The Specification has traditionally not accounted for long-term effects due to creep and shrinkage; as such, the stiffness prescribed in this section was developed based on studies examining only short-term behavior. Refer to Commentary Sections I1 and I3.2 for additional discussion.

Based on the above information, the approach described in Section I1.5 of AISC is considered to be acceptable, except that the caveats in the second and third paragraphs above (taken from the Commentary of Section I1.5 of AISC 360-16) should be addressed when applying the direct analysis method of design for structural systems that include encased composite members or filled composite members.

4.11 Chapter NJ Design of Connections

4.11.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Section NJ1, General Provisions, includes all of J1 of AISC 360, except Section J1.9 is replaced with the following: "9. Rivets - Rivets shall not be used in safety-related nuclear facilities." Also, Section J1.10 of AISC 360 is replaced with a list identifying the connections where pre-tensioned high-strength bolts or welds shall be used. The provision of NJ1.9 is appropriate for nuclear construction. The provisions of NJ1.10 are almost the same as N690-1994 (R2004) with the addition of a note for vibrating machinery.

In general, the provisions of J1 of AISC 360-10 are similar to J1 of the 1986 LRFD Specification and Q1.15, Connections, of N690-1994 (R2004). Improvements have been noted and some differences and omissions have been observed. It was concluded that further review regarding the basis for these changes was not warranted.

Section NJ2, Welds, includes all of J2 of AISC 360, except changes to 2b, Limitations and 6, Filler Metal Requirements. The changes in NJ2.2b make the provisions in AISC 360-10 more restrictive. The NJ2.6 Commentary states that additional notch toughness requirements have been incorporated based on the "*Seismic Provisions*."

Section J2 of AISC 360-10 states that all provisions of AWS D1.1/D1.1M apply under this specification with the exceptions listed; however, the Commentary to AISC 360-10 does not discuss the basis for exceptions listed. Based on the review, it was concluded that the exceptions taken to AWS in Section J2 of the current spec are similar to the exceptions taken in the previously NRC endorsed N690-1994 (R2004). The provisions in AISC 360-10 are almost the same as in AISC 360-05. They are also similar to the 1986 LRFD Specification, with some updating.

Section NJ3, Bolts and Threaded Parts, includes all of J3 of AISC 360, except 8, High-Strength Bolts in Slip-Critical Connections, and 10, Bearing Strength at Bolt Holes. NJ3.8 made one addition to this section, which is an improvement. NJ3.10 has made some modifications that reduce the options given in AISC 360-10.

Section J3 in AISC 360-10 is like AISC 360-05 with some additions. It is expanded and updated compared the 1986 LRFD and N690-1994 (R2004) Specifications. It was concluded that further review of the details of the differences was not warranted since the current specification represents an evolution of the state of the art.

Sections NJ4 through NJ10 include all of Sections J4 through J10 of AISC 360, with no exceptions or additions. It was concluded that no specific issues have been identified that warrant further review of these sections.

4.11.2 N690-18 Comparison to N690-2012

A summary of the significant changes to Chapter NJ in N690-18 are discussed below.

1. N690-12 included changes to NJ1.10, Limitations on Bolted and Welded Connections. Apparently, these do not appear in N690-18 since Section J1.10 was deleted from AISC 360-16.
2. N690-12 included changes to NJ2.2b, Limitations. These are not included in N690-18. This appears to be acceptable based on the changes made to AISC 360-16.
3. N690-12 included changes to NJ2.6, Filler Metal Requirements. N690-18 now includes the following provision: Welds subject to impactive and/or impulsive loads shall be made with filler metals meeting the requirements specified in AWS D1.8/D1.8M, clauses 6.1, 6.2 and 6.3. The new provision is probably acceptable since both the old and new versions reference AWS. Further review to confirm this is not warranted.
4. New provisions are added to NJ3.1 which appear to be acceptable since they add additional requirements when bolts are pretensioned.
5. N690-12 included a change to NJ3.8, High-Strength Bolts in Slip-Critical Connections, which stated: All faying surfaces shall be prepared as required for Class A or better surfaces. It is not clear why this provision is not included in N690-18, but it did not appear to be warranted to pursue this issue further.
6. The changes in the N690-18 to NJ3.10 are consistent with N690-12 and appear to be acceptable.
7. The following new provision was added to N690-18, NJ3.13, Connections for Members Subject to Impactive or Impulsive Loads: Bolted connections for members that are subject to impactive or impulsive loads shall be configured such that a ductile limit state controls the connection design. This provision appears to be acceptable.

<u>Evaluation of Commentary to N690-18</u>

The N690-18 NJ Commentary includes discussion on NJ2.6, Filler Metal Requirements, and NJ3.13, Connections for Members Subject to Impactive or Impulsive Loads.

The Commentary in N690-12 on NJ1.10 was deleted from N690-18 with no explanation. This is consistent with the Spec change discussed above, which also had no explanation.

The Commentary in N690-12 on NJ2.6 is the same as in N690-18, even though there were changes to the Spec in N690-18. See discussion on Specification changes above.

The Commentary in N690-12 on NJ3.10 was deleted from N690-18. The changes to NJ3.10 are consistent in both documents. However, the User Note in NJ3.10 negates the need for the previous Commentary.

The Commentary for NJ3.13 appears to be appropriate for the change made to the Specification.

4.11.3 AISC 360-16 Comparison to AISC 360-10

AISC 360-16 revised the provisions for bolts in combination with welds.

The revised provisions pertain to Section J1.8 of AISC 360-16 and are explained in the Commentary. The change appears to be based on recent tests and analysis. Further review of this change is not warranted.

AISC 360-16 increased the minimum pretension for 1 1/8-in.-diameter and larger bolts.

Minimum bolt pretension is provided in Table J3.1. Section J3.1 of the Commentary states that the minimum bolt pretensions listed in Table 8.1 of the Specification for Structural Joints Using High-Strength Bolts (RCSC, 2014) are as listed in Table J3.1. There does not appear to be any explanation for the change. The change was only made to Group A (e.g., A325 bolts) and not to Group B (e.g., A490 bolts) and increased the pretension by about 14%. Also, Table J3.1M, which lists the minimum bolt pretension in SI units was not changed. It is noted that Group C (e.g., F3043 Gr 2 bolts) was added to Table J3.1. The Commentary provides a discussion of this change.

The change to A325 bolts is consistent with the change in strength to A325 bolts greater than 1" in previous standards. The strength change increased from 105 ksi to 120 ksi as reported in Section A3.3 of the Commentary. A change of about 14%.

It is noted in the A3.3 Commentary that the strength of A325M bolts increased from 725 MPa to 830 MPa. However, Table J3.1M was not changed.

Tables J3.1 and J3.1M were checked and found to be consistent. All pretension is based on the tensile strength of 120,000 psi in both tables and for all bolt diameter sizes.

Based on the above review, this change is acceptable.
AISC increased standard hole sizes and short-slot and long-slot widths for 1-in. diameter and larger bolts.

These changes pertain to Section J3.2 and are explained in the Commentary. Sufficient justification is provided for this change.

The significant changes to Chapter J were discussed above. Some additional changes to Chapter J were made. A new Section J1.9, which covered Welded Alterations to Structures with Existing Rivets or Bolts, was added. Section J1.10, Limitations on Bolted and Welded Connections, was deleted. Additions and/or revisions were made to Sections J2, J3, J9, and J10.

It is time consuming to assess whether the above changes are discussed in the Commentary. It is judged that a further review of these changes is not warranted.

4.12 Chapter NK Design of HSS and Box Member Connections

4.12.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter NK includes all of Chapter K of AISC 360, except the preamble's second User Note is replaced as follows:

"User Note: See also Chapter J of the Specification and as modified by Chapter NJ of the Nuclear Specification for additional requirements for bolting and welding to HSS material."

The Chapter NK replacement for the second User's Note of the preamble for Chapter K of AISC 360-10 is acceptable.

The subject of this Chapter is not specifically addressed in N690-1994 (R2004). A new section Q1.5.3.1, Design Wall Thickness of Hollow Structural Sections, was added to N690-1994 (R2004). This section addresses the design wall thickness to be used in calculations for electric-resistance welded (ERW) HSS and submerged-arc welded (SAW) HSS and appears to be the only section that specifically addresses HSS design in N690-1994 (R2004). This subject also is not addressed in the 1986 LRFD Specification. It was covered in AISC 360-05 and updated in AISC 360-10. Many tables (with equations) are used in AISC 360-10 versus text and numbered equations in AISC 360-05.

The AISC 360-10 Commentary states that the provisions of Chapter K are based on failure modes reported in international research on HSS sponsored by CIDECT (International Committee for the Development and Study of Tubular Construction) since the 1960s. This work also received critical review by the International Institute of Welding (IIW) Subcommission XV-E on Tubular Structures. The Commentary states that the HSS connection design recommendations are generally in accord with the recommendations by this Subcommission (IIW, 1989).

In summary, the subject of this Chapter was not addressed in N690-1994 (R2004). The provisions of this Chapter are based on international research by CIDECT and the recommendations of the International Institute of Welding (IIW) Subcommission XV-E on Tubular Structures. It was concluded that further review of this Chapter was not warranted.

4.12.2 N690-18 Comparison to N690-2012

No changes to Chapter NK.

4.12.3 AISC 360-16 Comparison to AISC 360-10

AISC reorganized the HSS connection design provisions in Chapter K, including reference to Chapter J for some limit states.

The Commentary does not identify the specific changes that have been made. A comparison with AISC 360-10 would need to be performed to identify any significant changes. However, as discussed in the Commentary, this section is based on international research and has been critically reviewed by IIW. Further review of this change is not warranted.

4.13 Chapter NL Design for Serviceability

4.13.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter NL includes all of Chapter L of AISC 360, except Section L1, General Provisions, is replaced with the following:

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

There is no comparable information in N690-1994 (R2004). The information in AISC 360-10 Chapter L is general and qualitative. There is nothing in AISC 360-10 Chapter L that relaxes any other design guidance contained in AISC 360-10.

The value of including this in N690-2012 is not obvious. However, inclusion does not create any technical issues requiring detailed evaluation and reconciliation.

4.13.2 N690-18 Comparison to N690-2012

Changes are the same as N690-12. No further review needed.

Evaluation of Commentary to N690-18

The Commentary in N690-18 is the same as in N690-12. There was no need to make changes since the Specification was not changed.

4.13.3 AISC 360-16 Comparison to AISC 360-10

In L1 of the Commentary, Items (2) and (4) to (6) were added to the list in the paragraphs below:

> The general types of structural behavior that are indicative of impaired serviceability in steel structures are:
>
> (1) Excessive deflections or rotations that may affect the appearance, function, or drainage of the building, or may cause damaging transfer of load to nonstructural components and attachments
> (2) Excessive drift due to wind that may damage cladding and nonstructural walls and partitions
> (3) Excessive vibrations produced by the activities of the building occupants or mechanical equipment, that may cause occupant discomfort or malfunction of building service equipment
> (4) Excessive wind-induced motions that may cause occupant discomfort
> (5) Excessive effects of expansion and contraction caused by temperature differences as well as creep and shrinkage of concrete and yielding of steel
> (6) Effects of connection slip, resulting in excessive deflections and rotations that may have deleterious effects similar to those produced by load effects
>
> In addition, excessive local damage (local yielding, buckling, slip or cracking) or deterioration (weathering, corrosion and discoloration) may also affect the function and serviceability of the structure during its service life.

Section L2, Camber, in AISC 360-10 was deleted. The sections on Deflections and Drift were shortened and provide general information. The previous section on Expansion and Contraction now specifically addresses Thermal Expansion and Contraction, and provides general information. The Commentary has a few other revisions in addition to those noted above.

In conclusion, this chapter does not warrant further review.

4.14 Chapter NM Fabrication and Erection

4.14.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Section NM1, Shop and Erection Drawings, replaces M1 of AISC 360. The first paragraph is the same as M1 of AISC 360. The first half of this paragraph is also like Q1.1.2 of N690-1994 (R2004). The second half added the discussion on erection drawings and some other improvements. The second paragraph is new and it refers to the Code of Standard practice; thus it does not warrant further review. The third paragraph is new and seems appropriate.

Section NM2, Fabrication, includes extensive changes (subsections 1, 2, 3, 4, 7, 9) and additions (subsections 12, 13, 14, 15) to M2 of AISC 360. There are similarities and many additions to N680-1994 (R2004). A more thorough review of other reference documents would be needed to track down all the identified differences. It was concluded that such an activity was not warranted since Chapter MN2 may simply represent updated current practice and older provisions may now be incorporated in other updated referenced standards.

Section NM3, Shop Painting, replaced Subsection 4, “Finished Surfaces,” of M3 of AISC 360.

NM3 is similar to Q1.24 in N690-1994 (R2004). However, Q.1.24.1 states “… all other steel work shall be given one coat of shop paint.” M3 (1) indicates it is not required unless specified. This change is probably not significant. Q1.24.1 refers to RG 1.54. This is covered in general by a new User Note. Also, the new Subsection 4 takes exception to stainless steel and adds approval by the engineer of record.

Section NM4, Erection, includes the replacement of Subsection 2 “Stability and Connections” of M4 of AISC 360. Also, Section NM4 adds a new Subsection 7, “Tolerances for Cranes” to M4 of AISC 360.

The other Subsections are similar to N690-1994 (R2004). The following subsections in N690-1994 (R2004) are not specifically addressed: Q1.25.2, Q1.25.7.1, Q1.25.7.2, Q1.25.7.4, Q1.25.7.5, and Q1.25.8.

A more thorough review of other reference documents would be needed to track down all the identified differences. It was concluded that this activity was not warranted since Section NM4 represents either updated current practice or older provisions are now incorporated in other updated referenced standards.

4.14.2 N690-18 Comparison to N690-2012

Many of the changes in N690-18 are the same as in N690-12. The following differences were noted during the review:

Section NM2.4, Welded Construction, is revised. The first line states, “Welding shall be performed in accordance with AWS D1.1/D1.1M and AWS D1.6/D1.6M except as modified in

Section J2.” The first line of NM2.4 is similar to N690-12. The remainder is new and appears to be acceptable.

Section NM2.7, Dimensional Tolerances, has an additional item (4) added with the following title: “Steel-Plate Composite (SC) Wall Panels.” This new item is approximately two pages in length and appears to have appropriate provisions that do not warrant further review.

Two additional sentences were added to NM3.4, Finished Surfaces. These new provisions appear to be acceptable.

Evaluation of Commentary to N690-18

The Commentary for N690-18 is the same as the Commentary for N690s1-15. N690s1-15 added material related to SC construction in sections NM2.4, NM2.7 and NM4.2. The other section of NM2.4, as well as NM3 and MN4.7 are also the same as N690-12.

4.14.3 AISC 360-16 Comparison to AISC 360-10

Chapter M has some small changes (M2.2, M2.4, M2.10, M3.5, and M4.3) which do not appear to be significant. M1 of Commentary, Shop and Erection Drawings, is short. The only change is to update reference to Code of Standard Practice from (AISC, 2010) to (AISC, 2016). M2.2 of Commentary, Thermal Cutting, was revised. These changes do not warrant further review.

4.15 Chapter NN Quality Control and Quality Assurance

4.15.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Sections NN1 through NN7 completely replace Chapter N of AISC 360. Section Q1.0.1 Scope of N690-1994 (R2004) referenced 10CFR50 Appendix B for Nuclear Power Stations. Supplement 2 expanded this to include reference to Independent Spent Fuel Facilities and DOE Nuclear Facilities. Supplement 2 also adds ANSI/ASME NQA-1 to the referenced Codes and Standards and discusses the use of this standard in CQ1.29 of the Commentary. The QC/QA information in Chapter NN of N690-12 says it replaces the information in N of AISC 360-10, but it does follow closely what is in N of AISC 360-10. No mention is made of 10CFR50 Appendix B nor ANSI/ASME NQA-1. Section NA2, References, does add NQA-1 as a reference and the Commentary for NN does discuss NQA-1 and NQA-1a. Section NA5 of the specification discusses Appendix B and NQA-1. It is recommended that Section NA5 and Chapter NN be reviewed by the NRC for consistency with current staff guidance.

4.15.2 N690-18 Comparison to N690-2012

Many of the changes in N690-18 are the same as in N690-12. The following differences were noted during the review.

In the preamble, the first User Note was revised to add the following regarding SC construction: “As noted in Section NN6, steel-plate composite (SC) construction designed in accordance with Appendix N9 shall comply with applicable provisions (for the concrete and concrete reinforcing steel) of ACI 349 or ACI 349M for tests, materials and construction requirements.”

Section NN4 had some changes to Items 1, 2 and 3. Item 1 had some slight changes and added a User Note. Items 2 and 3 had some provisions added related to stainless steel welding.

Section NN5.2, Quality Assurance, had some changes and a User Note added related to the interruption of the work by the QA inspector.

Section NN5.4 had some changes to User Notes. One User Note was previously included in Section NN5.5g.

Various changes were made to Section NN5.5, Nondestructive Examination of Welded Joints, which probably do not warrant further review.

Table NN5.6-1, Inspection Tasks Prior to Bolting, changed wording from "proper" to "correct" in three places.

Section NN5.7 was added to address "Inspection of Galvanized Structural Steel Main Members."

Section NN5.8, Other Inspection Tasks, has some revisions. Some previous provisions that provided examples prefaced with the term "such as" were converted into User Notes.

Section NN6, MINIMUM REQUIREMENTS FOR INSPECTION OF COMPOSITE CONSTRUCTION, had revisions to Table NN6.1 and the addition of a paragraph related to steel-plate (SC) composite walls.
Tables NN6.2 and NN6.3 are new and relate to the Inspection of SC Wall Prior to Concrete Placement and after Placement of Concrete.

The above revisions do not warrant further review.

<u>Evaluation of Commentary to N690-18</u>

The first five paragraphs of the Commentary for N690-18 are the same as N690-12 and N690s1-15, except for updates to the dates of the referenced standards. Two clarifying paragraphs were added to N690-18 and appear to be acceptable.

Section NN5 was added to the N690-18 Commentary that addressed Section NN5b, CJP and PJP Groove Weld NDE, and Section NN5.g, Documentation. These do not warrant further review.

The N690-18 Commentary for NN6 is the same as N690s1-15 (with a minimal editing) and the addition of the paragraph related to the source of various inspection attributes in Table NN6.1. This change appears to be acceptable.

Section NN7 was added to the N690-18 Commentary which addresses nonconforming material and workmanship which appears to be acceptable.

4.15.3 AISC 360-16 Comparison to AISC 360-10

Since Chapters NN1 through NN7 completely replace Chapter N of AISC 360, there is no need to review AISC 360-16.

4.16 Appendix N1. Design by Advanced Analysis

N690-12 Appendix N1 and AISC 360-10 Appendix 1 were titled "Design by Inelastic Analysis". They were replaced by N690-18 Appendix N1 and AISC 360-16 Appendix 1, titled "Design by Advanced Analysis".

AISC 360-16 Appendix 1 is organized as follows: 1.1 General Requirements, 1.2 Design by Elastic Analysis, and 1.3 Design by Inelastic Analysis. Only the assessment of N690-18 Appendix N1 and AISC 360-16 Appendix 1 is presented.

4.16.1 N690-18 Appendix N1 and AISC 360-16 Appendix 1

N690-18 Appendix N1 modifies AISC 360-16 Appendix 1, Section 1.3 “Design by Inelastic Analysis”.

APPENDIX N1 - DESIGN BY ADVANCED ANALYSIS

> *Modify Appendix 1 of the Specification as follows.*
>
> N1.3. DESIGN BY INELASTIC ANALYSIS
>
> 1. General requirements
>
> *Add the following to the end of the first paragraph*:
>
> It is permitted to have localized inelastic behavior due to thermally induced load effects only in individual beams or their connections provided that a nonlinear inelastic analysis of the associated structure demonstrates that the structure is able to maintain its global stability and structural integrity to withstand all other concurrently acting loads.
>
> [User Note: Unlike impulsive and impactive loads, which affect a single or a few structural members, the accident temperature load case generally affects a large portion, if not the entirety of a structure. Also, unlike the case of design for impulsive and impactive loads, where the affected members are a priori known and therefore selectively targeted for detailing in accordance with the requirements of Section NB3.14, the same approach is difficult to implement for the accident temperature load case (except for incorporating thermal load relieving features mentioned in the User Note for Sections NB2.5 and NB2.6). Accordingly, only localized inelastic response in individual beams is permitted as long as it will not adversely affect the structure’s ability to resist other loads (e.g., sustained gravity load and the design basis earthquake load, which are part of the governing extreme environmental and abnormal load combinations).]
>
> *Add the following as the last paragraph*:
>
> When inelastic analysis is used for design, attention shall be paid to the induced deflections of the structural steel member(s), as well as to the effects of such deflections on supported components such as piping, HVAC ducts and cable trays, to ensure that the components will be able to perform their intended functions.
>
> [User Note: Increased deflections resulting from the utilization of inelastic design may cause additional component loading and may reduce component clearances (gaps) required to prevent vibration interaction.]

The first User Note references N690-18 NB3.14, which addresses design for impactive and impulsive loading using inelastic analysis.

The following information related to use of inelastic analysis is contained in NB3.

NB3. DESIGN BASIS

Add the following section:

[NB3.]14. Design Based on Ductility and Local Effects

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

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

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

For both options (a) and (b), the associated connections shall be designed such that their available strengths are greater than R_y times the nominal strength for LRFD and R_y/1.5 times the nominal strength for ASD of the connected member, where R_y value corresponds to the material used in the connected member and is obtained from *Seismic Provisions* Table A3.1.

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

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

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

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

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

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

[NB3.]3. Required Strength

Replace section [AISC 360-16 B3.3] with the following:

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

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

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

Previous editions of N690 did not have a Commentary for Appendix N1. The following was added to N690-18:

Modify Appendix 1 of the Specification Commentary as follows.

N1.3. DESIGN BY INELASTIC ANALYSIS

1. General requirements

Add the following to the end of the first paragraph:

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

4.16.2 Evaluation

Currently, the staff accepts the use of inelastic analysis methods, specifically applied to impulsive and impactive loads generated by pipe breaks or missiles. This provision is in N690-1994 (R2004), which was previously accepted by the staff.

The current SRP 3.7.2.II.8 also allows limited use of inelastic analysis for Seismic II/I evaluations. Seismic Category II SSCs may be important to safety if their collapse or excessive displacement has the potential to negatively impact the intended function of safety-related SSCs. Important to safety Category II SSCs are allowed to go beyond linear elastic limits, provided (1) there is ample clearance to accommodate the increased deflection, and (2) structural integrity is assured

The current SRP does not recognize the use of inelastic analysis for the overall structural response of safety-related Seismic Category I SSCs due to seismic loading. However, it is important to differentiate between analysis to determine seismic demand on the steel structures, and analysis to demonstrate design adequacy for all defined load combinations. Current staff guidance is that seismic demand on structures is calculated based on linear elastic analysis. For reinforced concrete structures, demonstration of design adequacy is also based on linear elastic analysis. Structural stability is rarely a concern for thick-walled concrete structures; second-order effects are not typically investigated. However, for steel frame structures, evaluation of structural stability is mandatory. In conducting structural stability analyses, seismic horizontal and vertical loads must be included. A direct second-order nonlinear analysis may be performed to demonstrate design adequacy.

Three types of loads unique to commercial nuclear power plant design are (1) design basis accident temperature; (2) jet impingement and pipe whip loads from postulated pipe breaks; and (3) internal missiles generated by postulated rotating machinery failures. For all three, N690-18 permits the use of inelastic analysis methods to demonstrate an adequate safety margin.

The application of nonlinear elastic and inelastic analysis is addressed in several different sections of N690-18, with cross-references between them.

Accident temperature is addressed in Appendix N1, in the first addition to Section 1.3 – Design by Inelastic Analysis. The guidance and acceptance criteria are technically sound and reasonable. BNL considers it acceptable.

Impulsive and impactive loading is addressed in Section NB3.14 “Design Based on Ductility and Local Effects.” The guidance and acceptance criteria are sufficiently prescriptive, technically sound, and reasonable. BNL considers it acceptable.

Structural stability is a universal concern for steel frame building structures and is covered by N690-18 Chapter NC – Design for Stability [AISC 360-16 Chapter C and referenced Appendices 6, 7, and 8]. BNL provided its recommendations for design for stability in a separate assessment of N690-18 Chapter NC.

AISC 360-16 Appendix 1, Section 1.2, Design by Elastic Analysis, includes numerous references to the guidelines and procedures in AISC 360-16 Chapter C, Design for Stability. Direct second-order nonlinear elastic analysis, including geometric imperfections and stiffness reduction, is the preferred method for structural stability analysis. Appendix 1, Section 1.2 expands the guidance for implementation of direct second-order nonlinear elastic analysis. BNL finds the additional guidance to be complimentary to and consistent with Chapter C, and considers AISC 360-16 Appendix 1, Section 1.2, Design by Elastic Analysis, to be acceptable.

4.16.3 Recommendations

BNL makes the following recommendations, related to the use of nonlinear elastic and inelastic analysis methods:

(1) Nonlinear Inelastic Analysis, per N690 Section NB3.14, is acceptable for treatment of impactive and impulsive loads.

(2) Nonlinear Inelastic Analysis, per the N690-18 Appendix N1, Section N1.3.1 supplement to AISC 360-16 Appendix 1, Section 1.3.1, is acceptable for treatment of thermally induced local inelastic effects.

(3) Nonlinear Inelastic Analysis, per AISC 360-16 Appendix 1, Section 1.3, is acceptable for structural evaluation of Seismic Category II steel structures to address Seismic II/I. The details are subject to staff review each time this method is used.

(4) Nonlinear Elastic Analysis, per AISC 360-16 Appendix 1, Section 1.2, is acceptable for use with AISC 360-16 Chapter C for elastic stability analysis of safety-related steel building structures, in cases where stresses are below the elastic limit at the onset of instability.

(5) Nonlinear Inelastic Analysis, per AISC 360-16 Appendix 1, Section 1.3 is acceptable for use with AISC 360-16 Chapter C for inelastic stability analysis of safety-related steel building structures, in cases where stresses exceed the elastic limit prior to instability.

4.17 Appendix N2. Design for Ponding

4.17.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Appendix N2 includes all of Appendix 2 of AISC 360 with no changes. Section 2.1 is very similar to the 1986 LRFD Specification, Section K2 and Section 2.2 is very similar to Appendix K2 of the 1986 LRFD Specification. Section K2 and Section Q1.13.2 of N690-1994 (R2004) are almost the same. Section K2 refers to Appendix K2 for an alternate determination of flat roof framing stiffness which appears to be covered in a similar manner as discussed in the N690-1994 (R2004) Commentary. N690-1994 (R2004) limits total bending stress to .80Fy. This limit is not explicitly stated in the 1986 LRFD Specification, but the equations used seemed to imply the same limit. N690-1994 (R2004) states that stresses due to wind or seismic forces need not be included in a ponding analysis. This provision is not specifically included in the 1986 LRFD Specification, nor the latest AISC 360-10. However, the stress indices defined in Section 2.2 (and Appendix K2) are determined based on the stress due to nominal dead load and nominal load due to rainwater, thus seismic or wind stresses are not considered in the calculation.
It is noted in Section 2.2 of AISC 360-10, that the stress f_o is due to D+R, while in Appendix K2 of the 1986 LRFD specification, the stress f_o is due to 1.2D+1.2R. However, AISC 360-10 uses a resistance factor of 0.8 while the 1986 Specification uses a factor of 0.9. Thus, there does not

appear to be a significant difference. Other compensating factors may also occur between the two specifications to account for the change.

In essence Appendix 2 of AISC 360 is very similar to previous provisions of N690-1994 (R2004) and the 1986 LRFD Specification. No specific issues have been identified that warrant further review of Appendix 2.

4.17.2 N690-18 Comparison to N690-2012

No changes to Appendix 2 of the Specification.

4.17.3 AISC 360-16 Comparison to AISC 360-10

The introduction is revised as follows:

> This appendix provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding. <u>These methods are valid for flat roofs with rectangular bays where the beams are uniformly-spaced and the girders are considered to be uniformly loaded.</u>
>
> The appendix is organized as follows:
>
> 2.1. Simplified Design for Ponding
> 2.2. Improved Design for Ponding
>
> <u>The members of a roof system shall be considered to have adequate strength and stiffness against ponding by satisfying the requirements of Sections 2.1 or 2.2.</u>

The underlined sentences above were added to Appendix 2 for AISC 360-16. In Section 2.2, the definition of f_o was modified to include other loads acting concurrently. There were minor editorial changes and a User Note was deleted. There do not appear to be any significant changes to the Appendix 2, Design of Ponding, other than the one noted below.

The Commentary for Chapter B3.9 states:

> Previous editions of this Specification suggested that ponding instability could be avoided by providing a minimum roof slope of 1/4 in. per ft (20 mm per meter). There are cases where this minimum roof slope is not enough to prevent ponding instability (Fisher and Pugh, 2007). This edition of the Specification requires that design for ponding be considered if water is impounded on the roof, irrespective of roof slope. Camber and deflections due to loads acting concurrently with rain loads must be considered in establishing the initial conditions.

Further review of Appendix 2 is not warranted.

4.18 Appendix N3. Design for Fatigue

4.18.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

N690-12 Appendix N3 incorporates all of 360-10 Appendix 3 by reference, with no changes.

AISC 360-10 Section B3.11 “Design for Fatigue” states that fatigue shall be considered in accordance with Appendix 3, Design for Fatigue. This section is like K3 of the 1999 LRFD Specification and Q1.7.1 and Q1.7.2 of N690-1994 (R2004). All of the specifications refer to an appendix and state that fatigue need not be considered for seismic effects or for the effects of wind loading. AISC 360-10 Appendix 3 has a user note that refers to the AISC *Seismic Provisions* for structures subject to seismic loads.

The AISC 360-10 Appendix 3 Commentary states that fatigue resistance has been derived from an exponential relationship and a family of fatigue resistance curves. These relationships were established based on an extensive database developed by Keating and Fisher in 1986. The Commentary further states that prior to the 1999 LRFD Specification, stepwise tables meeting the above criteria of cycles of loading, stress categories, and allowable stress ranges were provided in the Specifications. A single table format was introduced in the 1999 LRFD Specification that provides the stress categories, ingredients for the applicable equation, and information and examples including the sites of concern for potential crack initiation. Further explanation is provided in the 1999 LRFD Commentary. The current AISC 360-10 Commentary also states that a detail not covered before 1989 was added to the 1999 LRFD Specification, to cover tension-loaded plate elements.

AISC 360-10 Appendix 3 appears to be very close to Appendix K3 of the 1999 LRFD Specification. There are some additions, which appear to be improvements. Table A-3.1, Fatigue Design Parameters appears to be the same as Table A-K3.1 of the 1999 LRFD Specification.

N690-94 (R2004) Appendix QB “Fatigue”, including the tables, is similar to the 1986 LRFD Specification. The 2004 Supplement to N690-94 did not incorporate the changes made in 1999 to the LRFD Specification. There is no explanation for not including the changes in the 2004 Supplement to N690-1994.

4.18.2 N690-18 Comparison to N690-2012

No changes.

4.18.3 AISC 360-16 Comparison to AISC 360-10

Section 3.1, General Provisions, was revised. Several paragraphs were re-ordered and additional information provided. The equations in Section 3.3, Plain Material and Welded Joints, were reformatted and a User Note added. Revisions were also made to Sections 3.4 and 3.5. A new Section 3.6, Nondestructive Examination Requirements for Fatigue was added. It is relatively short and states that for CJP groove welds, the maximum allowable stress range calculated by Eq A-3-1 applies only to welds that have been ultrasonically or radiographically tested and meet certain sections of AWS D1.1. This provision was originally in Section 3.1 of AISC 360-10. Several revisions were made to Table A-3.1. The Commentary was expanded to seven pages in 360-16, from four pages in 360-10. The changes do not have any regulatory impact.

Background

The development of fatigue cracks and the consequential reduction in load carrying capacity is a concern for both safety-related steel structures and important-to-safety steel structures. Safety-related steel structures perform a direct safety function; e.g., support for or protection of safety-related SSCs. Important-to-safety steel structures are those whose collapse or excessive displacement would negatively impact the intended function of safety-related SSCs, even though the structure itself performs no safety-related function.

From a design perspective, the acceptance criterion must be no crack initiation due to cyclic loading. As the magnitude of cyclic loading increases, the number of cycles that can be tolerated decreases. For very large number of cycles ($>10^6$), local alternating peak stresses (stress range divided by 2) must remain below the threshold limit. At the threshold limit, an infinite number of stress cycles can be tolerated with no crack initiation. When local alternating peak stresses exceed the threshold limit, the design allowable number of stress cycles has an upper limit.

For critical pressure-retaining mechanical components in nuclear facilities, ASME Code Section III provides a formalized procedure for ensuring adequate resistance against fatigue cracking and considers cumulative fatigue damage when there are multiple cyclic loading scenarios.

<u>Detailed Assessment of AISC 360-16 Appendix 3</u>

AISC 360-16 Appendix 3 utilizes a different approach from ASME for ensuring adequate resistance against fatigue cracking. The nominal design allowable strength is unreduced if a structural member is subject to less than 20,000 total stress cycles over its design life. Above 20,000 cycles, adjustments are specified. There is no requirement to calculate local peak stress at fatigue-critical locations. AISC 360-16 specifies a maximum allowable nominal stress range, as a function of member and/or joint geometry, stress type and location, and the number of stress cycles.

N690-18 incorporates in its entirety the provisions of AISC 360-16 Appendix 3 – Design for Fatigue. There are many similarities and several differences between N690-1994 (R2004) Appendix QB - Fatigue and AISC 360-16 Appendix 3. The most significant difference is the specification of the maximum allowable stress range as a function of the number of cycles. Appendix QB includes Table QB1, which defines 4 cyclic loading conditions (20,000 to100,000; 100,000 to 500,000; 500,000 to 2,000,000; and >2,000,000). Table QB3 then defines a single allowable stress range for each cyclic loading condition.

As an example, in Table QB3, for Stress Category A (base metal, no stress risers, tensile or reverse loading) and Loading Condition 1 (20,000 to 100,000 cycles), the allowable stress range is 63 ksi.

In AISC 360-16 Appendix 3, equations are provided that define a unique allowable stress range as a function of the number of cycles. Appendix 3 represents a relaxation from Appendix QB. From AISC 360-16 Appendix 3, for Stress Category A [C_f = 25; F_{th} = 24 ksi]

$$F_{SR} = 1{,}000\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \qquad \text{(A-3-1)}$$

where

C_f = constant from Table A-3.1 for the fatigue category

F_{SR} = allowable stress range, ksi (MPa)

F_{TH} = threshold allowable stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa)

n_{SR} = number of stress range fluctuations in design life

Substituting n_{SR} = 20,000 in Eqn. (A-3-1), yields F_{SR} = 108 ksi.

For n_{SR} = 60,000, F_{SR} = 75 ksi.

For n_{SR} = 100,000, F_{SR} = 63 ksi.

Only at 100,000 cycles do Appendix QB and Appendix 3 match. Appendix 3 represents a relaxation from Appendix QB. This is a consistent pattern, with the allowable stress range only matching at the upper limit of the QB load condition range; i.e., 500,000, 2,000,000, >2,000,000.

BNL reviewed the technical basis for the current AISC 360-16 Appendix 3 provisions and also reviewed the history of the "Design for Fatigue" provisions in earlier AISC Specifications. The current provisions are essentially unchanged since the 1999 edition of "LRFD Specification for Structural Steel Buildings". They follow the recommendations in "Review of Fatigue Tests and Design Criteria on Welded Details", Keating, B. and Fisher, J. W., National Cooperative Highway Research Program (NCHRP) Project 12-15(50), October 1985, Washington, D.C. This document apparently was formally released as NCHRP Report 286 in September 1986. The "Design for Fatigue" provisions in N690-1994 (R2004) apparently incorporate earlier recommendations from the "AASHTO Standard Specifications for Highway Bridges". After reviewing the technical basis for the changes, BNL finds them acceptable.
AISC 360-16, Appendix 3, also contains the following text:

> "3.1. GENERAL PROVISIONS
> The provisions of this Appendix shall apply to stresses calculated on the basis of the applied cyclic load spectrum. <u>The maximum permitted stress due to peak cyclic loads shall be 0.66Fy</u>. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation."

This paragraph implies that the maximum allowable stress range is 2 x 0.66 Fy = 1.33 Fy, corresponding to complete stress reversal, and this limit supersedes the allowable stress ranges calculated in accordance with the Equation (A-3-1).

For a material with Fy = 60 ksi, the allowable stress range would be 80 ksi based on 1.33 Fy. This is significantly lower than the 108 ksi allowable stress range for 20,000 cycles, calculated by Eqn. (A-3-1). Therefore, when conducting a fatigue calculation for allowable stress range in accordance with Eqn. (A-3-1), the analyst must also be cognizant of the 1.33 Fy upper limit on allowable stress range.

Recommendations

After detailed review of AISC Specification 360-16, Appendix 3 - Design for Fatigue, BNL concludes that

(1) Relaxation of the allowable stress range versus number of loading cycles, compared to N690-1994 (R2004) Appendix QB, is acceptable.

(2) It is incumbent on the analyst to be aware that AISC 360-16 Appendix 3, Section 3.1 places a limit on the allowable stress range that may be lower than the value obtained from Eqn. (A-3-1).

4.19 Appendix N4 Structural Design for Fire Conditions

4.19.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Chapter N4.1, General Provisions, includes all of Appendix 4 of AISC 360, except the following paragraphs are added to Section 4.1:

> “The intended functions of the structure under design basis fire shall be stated in the licensing document. The provisions of Appendix N4 are for life safety associated with evacuation of building occupants in the event of a design-basis fire.
>
> This specification does not address either “Important to Safety” structural steel members or loading conditions associated with a facility fire.
>
> Structural steel shall be fire protected to achieve the fire resistance rating as established by fire hazard analysis. Where engineering analysis is used for structural design for fire conditions, design material parameters at elevated temperatures during the design-basis fire event shall be those defined in Table NA-4.2.1 and Table NA-4.2.2. Other material parameter values may be used provided they are substantiated or verified by test. The possible increased deflection that may occur due to elevated temperatures shall be considered in the design.”

N690-1994 (R2004) and earlier specifications (both ASD and LRFD) did not address structural design for fire conditions. Guidance on this subject was included for the first time in AISC 360-05, Appendix 4. N690-06 made no changes to Appendix 4 of AISC 360-05.

Appendix 4 to AISC 360-10 revised sections of AISC 360-05, added new information, and provided additional guidance in the Commentary. The strength design criteria at elevated temperatures has been revised to reflect recent research. Section 4.2.4.3b of the Commentary indicates that new equations were introduced to reduce unconservative errors in the 2005 Specification. The Commentary includes a two-page Bibliography to provide further information on key issues related to fire-resistant design.

It is noted that Section 4.1.2 states that structural design for fire conditions shall be performed using the LRFD method. The Commentary explains that it is difficult to develop design equations to ensure the necessary level of structural performance during severe fires using elastically based ASD methods.

It is not clear why Section N4.1 states that the Appendix does not address “Important to Safety” steel members. If “Important to Safety” structures are not covered by Appendix N4, it should be clarified as what nuclear structures are meant to be covered by this appendix. The remaining additional criteria in N4.1 appears to be appropriate.

Section B3.12 appears to make the guidance in Appendix 4 moot, although one could always choose to use it. B3.12 states: “Compliance with the fire-protection requirements in the applicable building code shall be deemed to satisfy the requirements of Appendix 4. It further states: “This section is not intended to create or imply a contractual requirement for the engineer of record responsible for the structural design or any other member of the design team.”

N690-12, Section N4.2. Structural Design for Fire Conditions by Analysis; N4.2.3.1. Thermal Elongation replaces Appendix 4, Section 4.2.3.1 with the following:

> “The coefficients of expansion shall be taken as follows:
>
> (a) For structural and reinforcing steels: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 7.8×10^{-6}/°F (1.4×10^{-5}/°C).
>
> (b) For normal weight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 5.5×10^{-6}/°F (9.9×10^{-6}/°C).
>
> [User Note: Table NA-4.2.1 is intended for carbon steel applications. For stainless steel and other alloy steels the user needs to establish appropriate values based upon testing or qualified references.]”

The replacements for Section 4.2.3.1 are the same as AISC 360-10. For (b) the coefficient is changed from 1.0×10^{-5}/°F (1.8×10^{-5}/°C) to 5.5×10^{-6}/°F (9.9×10^{-6}/°C). No explanation is given for this change. Item (c) in AISC 360-10 is for lightweight concrete and is omitted in N4.2, as well as Table A-4.2.2, with no explanation. Presumably this is because nuclear structures do not use lightweight concrete. The new user note in N.2 is appropriate.

Table NA-4.2.1 and Table A-4.2.2 are also modified.

Table NA-4.2.1 is modified to use ambient temperature properties for k_u up to 600°F. Table NA-4.2.2 deleted the column for lightweight concrete and added the following footnote: “At 1000°F, concrete starts to deteriorate rapidly and the strength of reinforcing steel will be affected. This shall be taken into account in the design.” No explanation is provided in the Commentary for these modifications. They do not appear to be significant.

As discussed above, Section N4.1 indicates that the Appendix does not address “Important to Safety” steel members. In addition, NRC has specific fire protection regulations in Title 10 of the Code of Federal Regulations (10 CFR) Section 50.48, "Fire Protection," parts (a) and (b), and 10 CFR Part 50, Appendix R, “Fire Protection Program for Nuclear Power Facilities Operating Prior to January 1, 1979,” as well as regulatory guidance in NRC Regulatory Guide 1.189 “Fire Protection for Nuclear Power Plants.” Therefore, it is recommended that Appendix N4 should not apply to nuclear power plants; however, NRC review of this issue is also recommended.

4.19.2 N690-18 Comparison to N690-2012

Appendix N4 in N690-18 has minor changes compared to N690-12.

Evaluation of Commentary to N690-18

The Commentary for N690-18 is the same as N690-12.

4.19.3 AISC 360-16 Comparison to AISC 360-10

AISC 360-16 inserted a table of properties of high-strength bolts at elevated temperatures in Appendix 4.

Section 4.2.3 of the Commentary provided sufficient justification for this change.

Some additional modifications were made and material was added to the Commentary. The changes made to Appendix 4 do not appear to warrant further review.

4.20 Appendix N5. Evaluation of Existing Structures

4.20.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Sections N5.1 through N5.5 of Appendix N5 completely replace Appendix 5 of AISC 360. The information in Appendix N5 is not in N690-1994 (R2004). Appendix N5 is similar to Appendix 5 of AISC 360-10, with some modifications. The Commentary to Appendix 5 of AISC 360-10 refers to ASCE/SEI 31 for earthquake evaluation of existing buildings and ASCE/SEI 41 seismic rehabilitation work. This was deleted from the Commentary to N5, which appears appropriate. However, the deletion of this information now makes the introduction to the Commentary to N5 seem a little contradictory. As written the standard does not address seismic and other dynamic loads and therefore would not appear to be applicable to nuclear facilities. The endorsement of N690-12 should take exception to Appendix N5 since it does not address seismic and other dynamic loads.

4.20.2 N690-18 Comparison to N690-2012

The preamble to Appendix N5 was revised as follows

> *"Replace Appendix 5 of the Specification with the following:*
>
> This appendix applies to the evaluation of the strength and stiffness under static loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section NA3.1. This appendix does not address load testing for the effects of seismic and other dynamic loads. Section N5.4 is only applicable to static vertical gravity loads applied to existing roofs or floors.
>
> User Note: The scope of Appendix N5 follows AISC 360. Where the evaluation is for existing safety-related structures subjected to other than static loads or load combinations, or where the evaluation uses dynamic load analysis, dynamic testing or load tests other than those in the scope of Section N5.4, the engineer of record is responsible to show that the test and analytical evaluation methods employed are acceptable to the AHJ."

In N690-12, the first line stated "….under static vertical (gravity) loads…" However, the last sentence was added in N690-18. This change appears to be acceptable.

Section N5.2.2, Tensile Properties, was revised and added a User Note. This revised section appears to be improved.

Changes were made to the last two paragraphs of N5.4.1 and to N5.4.2. These changes appear to be acceptable. The most significant change is as follows: "It shall be demonstrated, while maintaining the maximum test load for one hour, that the deformation of the structure does not increase by more than 10% above that at the beginning of the holding period."

The underlined portion originally stated: "remains essentially unchanged." This seems to be a more realistic expectation.

Evaluation of Commentary to N690-18

In the first line of N5.1, the previous issue referred to "gravity loading," not "static loading." The use of the word "static" in lieu of "gravity" does not appear to be correct.

Section N5.2.2 is new and would need to be reviewed further if it is decided not to take exception to this appendix.

In the first line of N5.2.6, the words "required to be" were added. The change appears to be appropriate.

4.20.3 AISC 360-16 Comparison to AISC 360-10

There does not appear to be any significant changes to Appendix 5. Some small revisions were also made to the Commentary. This appendix does not warrant any further review.

4.21 Appendix N6. Stability Bracing for Columns and Beams

There are no provisions in N690-1994 (R2004) for designing stability bracing. N690-18 incorporates by reference all of AISC 360-16 Appendix 6. This is a major addition to the specification for nuclear facilities. The provisions in Appendix 6 of AISC 360-16 have evolved over time based on industry practice.

4.21.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Appendix N6 incorporates by reference all of Appendix 6 of AISC 360-10, with no changes. There is no comparable section in N690-1994 (R2004).

4.21.2 N690-18 Comparison to N690-2012

No changes to Appendix N6, except that AISC 360-16 replaces AISC 360-10.

4.21.3 AISC 360-16 Comparison to AISC 360-10

Appendix 6 – Member Stability Bracing

This appendix was previously titled "Stability Bracing for Columns and Beams." The User Note in the introduction has been changed as follows:

> Stability requirements for lateral force-resisting systems are provided in Chapter C. The provisions in this appendix apply to bracing that is not generally included in the analysis model of the overall structure, but is provided to stabilize individual columns, beams and beam-columns. Guidance for applying these provisions to stabilize trusses is provided in the Commentary.

An introduction was added to the Commentary as follows:

> This Commentary provides background to the development of the Appendix 6 equations and explains their application in the design for bracing of beams, columns and beam-columns.
>
> In the design of bracing for trusses, the compression chord may be treated as the compression flange of a beam. Further discussion of specific bracing applications for trusses and other systems can be found in this Commentary.

While a cursory review of Appendix 6 was performed, a more detailed evaluation was not warranted. The design of stability bracing for nuclear facilities following the provisions of Appendix 6 of AISC 360-16 is considered acceptable, based on engineering judgment.

4.22 Appendix N7. Alternative Methods of Design for Stability

In addition to the information below, see evaluation for Chapter NC.

4.22.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Appendix N7 includes all of Appendix 7 of AISC 360 with no changes.

4.22.2 N690-18 Comparison to N690-2012

No changes to Appendix N7.

4.22.3 AISC 360-16 Comparison to AISC 360-10

Appendix 7 – Alternative Methods of Design for Stability

There was a small clarification added in Section 7.2.1 and two changes were made in Sections 7.2.3 and 7.3.3 related to effective length. Section 7.3.1(c) was revised to address nonslender and slender-element sections. There does not appear to be any other changes. Some changes were made to the Commentary. These revisions do not warrant further review.

4.23 Appendix N8. Approximate Second-Order Analysis

In addition to the information below, see evaluation for Chapter NC.

4.23.1 N690-12 and AISC 360-10 Comparison to N690-1994 (R2004)

Appendix N8 includes all of Appendix 8 of AISC 360 with no changes.

4.23.2 N690-18 Comparison to N690-2012

No changes to Appendix N8.

4.23.3 AISC 360-16 Comparison to AISC 360-10

Appendix 8 – Approximate Second-Order Analysis

One term in Eq A-8-5 was revised (K_1L is now L_{c1}). The User Note at end had one additional sentence as follows:

> R_M can be taken as 0.85 as a lower bound value for stories that include moment frames, and R_M = 1 if there are no moment frames in the story.

There do not appear to be any other changes. There were some small changes in the Commentary, including changes to two of the figures. These revisions do not warrant any further review.

4.24 Appendix N9 Steel-Plate Composite (SC) Walls

Appendix N9 is a new appendix introduced in N690s1-15, which provides specifications for SC walls in safety-related nuclear facilities. SC walls are considered modular type members constructed from steel faceplates spaced apart, connected with steel ties, and then filled with concrete. The faceplates also utilize steel anchors welded to the plates which, along with the ties, make the entire section act monolithically. Since there are no comparable provisions in AISC 360-16 Specification, Appendix N9 provides a complete set of provisions without reference to AISC 360. In addition, SC walls were not covered in N690-1994 (2004) so there is no comparison made to the prior N690 specification. The information presented below is based on the current N690-18 Specification.

4.24.1 Section N9.1 Design Requirements

This section of Appendix N9 provides provisions for the geometric configuration and material properties, design basis, faceplate slenderness requirement, requirement for composite action, steel tie requirements, design for impactive and impulsive loads, and design and detailing requirements around openings.

General Design Criteria

For geometric and material properties, provisions are presented which identify the minimum and maximum: section thickness, faceplate thickness, and reinforcement ratio. For material properties, the provisions identify the minimum and maximum: faceplate yield stress and compressive strength of concrete. The minimum thickness for exterior walls is based on the NRC SRP to resist tornado missile. Many of the provisions regarding geometric and material properties can be considered acceptable on the basis of experimental tests conducted in Japan, Korea, and the U.S.

N690-18 Section N9.1.1(d) of the Commentary states: “A minimum yield stress of 50 ksi (350 MPa) is specified for the faceplates to prevent: (i) residual (locked-in) stresses from concrete casting, and …” This statement along with discussions in a meeting with the N690 Task Committee (TC) -11 raised a question about how the design of SC walls consider the effects of the pressure load due to the concrete pour. As a result of the discussion, it was noted that Section NM2.7, Item (d) provides dimensional tolerances for SC wall panels as measured in the fabrication shop. This section also identifies that the dimensional tolerances for SC modules after concrete curing shall be governed by the concrete construction tolerances defined in ACI

349-13 or ACI 349M-13 and ACI 117-10 or ACI 117M-10. In addition, after the concrete cures, the faceplate waviness, f_v must be limited to a value given in Equation NM2-1 of N690-18. The Commentary to Section NM2.7 describes the use of finite element models to benchmark the effect of faceplate waviness on SC walls. In addition, a User Note in Section NM.2.7 states that the engineer of record may specify the concrete pour rate and height to meet the faceplate waviness requirements. On the basis of the above information, this question about the effect of the concrete pour on the design of SC walls is adequately addressed.

Section N9.1.2b of N690-18 - Design for Stability and the User Note refer to ACI 318 for conditions that should be met in order to avoid performing second-order stability analyses of SC walls. As discussed previously in Section 4.10 of this report, ACI 349 should be utilized, rather than ACI 318, since ACI 349 is applicable to nuclear safety-related concrete structures. Also, other sections of N690-18 (e.g., Section NA2 of N690-18) refer to AISC 360-16, which in turn refer to ACI 318-14 (2014) in some cases. Therefore, it is recommended that the provisions in ACI 349-13 and RG 1.142 Rev. 3 be followed, rather than ACI 318, unless otherwise justified.

The design basis of the SC walls considers the walls to be divided into interior regions and connection regions. In the case of the connection regions, requirements are specified for the minimum and maximum widths allowed corresponding to the SC section thickness and twice the SC section thickness, respectively. Further details for the design of SC wall connections are presented in Subsection N9.4 of N690.

The design basis provisions indicate that the required strength for SC walls and their connections shall be determined using an elastic finite element analysis for the applicable load combinations, except those associated with impulsive loads which are discussed in Section N9.1.6c. This approach is considered to be acceptable because it is consistent with the current SRP criteria and has been used in the past for such SC structural members which were found to be acceptable.

Provisions are presented for faceplate slenderness which require faceplates to be anchored to the concrete using steel anchors, ties, or a combination of these. Local buckling of the faceplates is considered to be one of the SC specific limit states. The provisions for slenderness are to ensure that the faceplate will yield in compression before local buckling. This has been demonstrated in experimental studies, and thus, is acceptable.

To ensure that the entire SC section acts as a composite member, provisions are specified regarding the potential for slippage of the anchors in shear and spacing of the anchors. For anchors that are classified as yielding steel anchors, the available shear strength is obtained based on the AISC 360 Specification. According to the Commentary, based on experimental testing, steel headed stud anchors are typically able to meet the criterion for yielding anchors. Other types of steel anchors need to be tested to determine their available shear strength and interfacial slip capability. The classification requirement for the steel anchors is considered to be acceptable because it is based on experimental testing.

Provisions are provided for spacing of the anchors to develop the yield strength of the faceplates and to prevent interfacial shear failure. In the case of interfacial shear failure, the provisions for the spacing requirements for steel anchors prevent interfacial shear failure from occurring before out-of-plane shear failure. This ensures that interfacial shear failure, which is another SC specific limit state, will not occur. The equations for the spacing requirements for both developing yield strength of the faceplates and preventing interfacial shear failure are considered to be acceptable on the basis of the derivations provided in the Commentary for Section N9.1.4b.

Ties between the two faceplates are needed to contribute to the out-of-plane shear strength and enhance the structural integrity of the SC walls. The ties provide structural integrity by preventing delamination or splitting failure within the section which may occur due to load eccentricities. The equation in Section N9.1.5b identifies the required tensile strength for the individual ties due to this load eccentricity. The derivation of this equation is presented in the corresponding section in the Commentary. Since this derivation demonstrates the structural integrity of the section under the eccentric loading within the section, the equation is considered acceptable. Note that the design of the ties also requires it to satisfy the out-of-plane shear strength requirements specified in Section N9.3.5.

<u>Design for Impactive and Impulsive Loads</u>

N690-18 Section N9.1.6, Design for Impactive and Impulsive Loads, and its subsections identify the criteria and provisions for design of SC walls for impactive and impulsive loads. This section presents the required design criteria and ductility ratios for flexure-controlled SC walls, shear-controlled SC walls and SC walls under axial compressive loads. As explained in the Commentary to Section N9.1.6, the section is based on ACI 349-13, Appendix F - Special Provisions for Impulsive and Impactive Effects. While the criteria and provisions in N690-18 Section N9.1.6 are specific to SC walls, some of the criteria in this section of N690-18 are not consistent with those in ACI 349-13 Appendix F and NRC Regulatory Guide 1.142 Rev. 3 for reinforced concrete members. The Attachment to this report identifies the differences between the criteria and provisions in N690-18 Section N9.1.6, ACI 349-13 and RG 1.142 Rev. 3. The Attachment also identifies recommendations for regulatory positions in order to provide consistency between the guidance for the design of SC walls and RC walls subjected to impactive and impulsive loads. These recommendations are described below.

Recommendations for AISC N690-18, Section N9.1.6 - Design for Impactive and Impulsive loads

N690-18 Section N9.1.6 and its subsections are based on ACI 349-13, Appendix F, Special Provision for Impulsive and Impactive Effects adapted to SC walls. The provisions in Section N9.1.6 and its subsections are considered acceptable with the additions and exceptions identified below.

1. Impactive and impulsive loads are assumed to be concurrent with other loads (e.g., dead and live loads) in determining the required strength of structural elements.

2. For impulsive loads, the strength available for impulsive loads is at least 20 percent greater than the magnitude of any portion of the impulsive loading, which is approximately constant for a time equal to or greater than the first fundamental period of the structural member. See Table 6 in the Attachment for further details.

3. In addition to the deformation limits under items 4. to 7. below, the maximum deformation shall not result in the loss of intended function of the structural wall nor impair the safety-related function of other systems and components.

4. For flexure-controlled SC walls as defined in Section N9.6b of ANSI/AISC N690-18, the permissible displacement ductility ratio demand should satisfy all of the following:

- ductility ratio less than or equal to 10,
- principal strain of the faceplates less than or equal to 0.05 (Johnson et al., 2014), and
- rotational capacity of any yield hinge less than or equal to 0.07 radians (4 degrees) [Bruhl et al., 2017).

5. For SC walls resisting axial compression, the permissible displacement ductility ratio should be as shown in items 5.1 to 5.3.

    5.1 When compression controls the design as defined by the balanced point in a load-moment interaction diagram, the permissible ductility ratio shall be 1.0.

    5.2 When the compression load does not exceed 0.1(f'_c A_g), where A_g is the sum of the area of concrete infill and the net area of the faceplates, or one-third of that which would produce balanced strain conditions, whichever is smaller, the permissible ductility ratio should be as given in 4.

    5.3 The ductility ratio varies linearly between 1.0 and that given in 4. for conditions between those described in 5.1 and 5.2.

6. The permissible displacement ductility ratio in flexure should not exceed 3.0 for loads such as blast and compartment pressurization, which could affect the integrity of the structure as a whole.

7. For shear-controlled SC walls, with yielding reinforcement spaced at section thickness divided by two or smaller, the ductility ratio is limited to 1.3. For shear-controlled SC walls with other configurations of yielding or nonyielding reinforcement, the ductility ratio is limited to 1.0.

8. The shear strength under local loads considers reaction shear at the supports and punching shear adjacent to the load.

    8.1 Local loads may be impulsive or impactive, except that for impactive loads, satisfaction of criteria for perforation shall be used in place of punching shear requirements.

    8.2 The shear strength is determined in accordance with the provisions in N690 Section N9.3, Design of SC Walls, using the appropriate dynamic increase factors (DIFs) in Table A-N9.1.1 for the required concrete and steel material properties.

    8.3 In the case of the reaction shear (beam action condition) at the supports, the effective width of the critical section for the shear beam capacity at the supports is to be determined according to the zone of influence induced by the local loads instead of the entire width of the support. The zone of influence induced by the concentrated loads may be determined, for example, by an analysis [see Lantsoght, Eva O. L., (2012)].

9. Design of SC walls or SC structural wall systems for impactive loads satisfies the criteria for local effects and overall structural response. Their structural response is determined by the methods for impulsive loads in Section N.9.1.6c. Local effects include penetration, perforation and punching shear. The penetration depth and required concrete and faceplate thickness required to prevent penetration are from applicable rational methods or pertinent test data.

10. Evaluation of loads from malevolent, beyond-design-basis, aircraft impacts are in the scope of RG 1.127 and outside the scope of this report.

Design and Detailing Around Openings

Design and detailing requirements around openings in SC walls are provided in N690-18 Section N9.1.7. Specific provisions are presented for small openings and large openings. For small openings, one option is to use a sleeve (steel plate) at the edge of the opening which connects the two faceplates. Another option is to reduce the section capacity of the affected SC panel sections by a specified amount where the opening exists. For large openings, a conservative analysis approach is described where the opening is required to be modeled in the analysis much larger than the actual opening, thereby ignoring the strength provided in the region around the opening that is not modeled. Alternatively, the edge can be considered fully developed with the same detailing requirements as the small opening. The provisions for the different options are based on available literature that includes testing and in other cases on utilizing the wall section strength only after some length (development length) away from the opening which is specified in Section N9.1.4b. Thus, the provisions for design and detailing around openings in SC walls are acceptable.

4.24.2 Section N9.2 Analysis Requirements

Section N9.2.1 provides general provisions for the analysis of SC walls. It indicates that the SC walls shall be analyzed using elastic, three-dimensional, thick-shell, or solid finite elements. Provisions are also provided for consideration of second-order effects if applicable, analysis involving accident thermal conditions, and damping ratio for use under the safe shutdown earthquake (SSE). Under SSE, the damping shall not exceed 5% damping for determining the required strengths for SC walls. The selection of 5% damping for SSE is based on scale tests of a containment internal structure constructed from SC modules. The commentary corresponding to this section indicates that for the operating basis earthquake, damping lower than 5% (2 to 3%) should be used. The commentary also provides some recommendations on the finite element discretization which is based on ASCE 4-98. These provisions are considered to be acceptable based on consistency with the NRC SRP, industry practice, good engineering principles, and testing in the case of damping.

Flexural Stiffness

The effective flexural stiffness for use in analysis of SC walls is provided in Section N9.2.2(a). Equations for the effective flexural stiffness and effective shear stiffness are presented. As explained in the Commentary, experimental studies show that the uncracked composite flexural stiffness does not generally occur in SC walls. Therefore, the cracked transformed flexural stiffness of the SC walls are derived based on equilibrium of the forces in the concrete and steel faceplates. These equations were then calibrated to experimental test data to arrive at a simpler form given by Equation C-A-N9-4. This equation corresponds to ambient thermal conditions. For accident thermal conditions, when the temperature is greater than 150°F (65.6°C), complete (through-wall) concrete cracking is assumed, and thus, the flexural stiffness equal to only the steel faceplates, i.e., E_sI_s, is specified, where E_s is the modulus of elasticity of steel and I_s is the moment of inertial of the steel faceplates. For change in temperatures at the surface of the faceplates between 0° and 150°F (-17.8° to 65.6°C), the cracked transformed flexural stiffness, is reduced linearly until it equals the section stiffness of only the steel, E_sI_s. Since the derivation of the effective flexural stiffness has been verified and calibrated based on test data, this approach is acceptable.

Equation A-N9-8M in Section N9.2.2 provides the effective stiffness of an SC wall in metric form based on the temperature of the SC exterior surfaces in degrees Celsius. The equation utilizes a temperature of 83 degrees Celsius in the denominator which is not consistent with the use of 150 degrees Fahrenheit in Equation A-N9-8. This should be corrected. There are also inconsistencies in the numerical values of degrees Celsius versus Fahrenheit in the Commentary (e.g., Section N9.2.2) which should be corrected.

<u>Shear Stiffness</u>

The effective in-plane shear stiffness, for all load combinations that do not include accident thermal loading, is provided in Section N9.2.2(b). When the required membrane in-plane shear strength is less than or equal to the cracking threshold, the effective in-plane shear stiffness is the summation of the uncracked concrete shear stiffness and steel shear stiffness, i.e., G_cA_c + G_sA_s. If the required in-plane shear strength is greater than twice the cracking threshold, then the effective stiffness is the post-cracking shear stiffness. For required membrane in-plane shear strength between the cracking threshold and twice the cracking threshold, a linear interpolation is used. The equations presented in Section 9.2.2(b) were based on test data and a three-step model that was developed by Varma et al., (2011a).

For the effective in-plane shear stiffness for all load combinations that include accident thermal loading, Section N9.2.2(c) indicates that stiffness shall be equal to the expression given in Equation A-N9-12 under Section N9.2.2(b).

Since the various equations given for determining the effective in-plane stiffness are based on test data and rational engineering mechanics principles, they are considered to be acceptable.

<u>Geometric and Material Properties for Finite Element Analysis</u>

Since an elastic finite element model is used to model the SC walls, a single material model would be used. Section N9.2.3 specifies to use Poisson’s ratio, coefficient of thermal expansion, and thermal conductivity corresponding to concrete. These are specified because the concrete is expected to govern the displacement of the structure due to thermal loads.

The effective model thickness and the elastic modulus are to be determined such that the effective stiffness values for flexure and shear are satisfied. Then the material density can be determined based on the effective model thickness. These are acceptable because the effective model parameters result in equivalence with the actual SC wall properties.

N690-18 does not specify how to model the actual in-plane axial stiffness of the SC walls. Since only two modeling parameters (effective element thickness and effective elastic modulus) are used to represent the actual flexural and shear stiffness of the SC walls, the modeled axial stiffness of the SC walls may not match the actual axial stiffness of the wall. However, in view of the typically large through wall thickness of SC walls, this is not expected to have a significant effect on the overall response of the structure. This is somewhat analogous to the approach in defining axial rigidity of reinforced concrete shear walls (Table 3-2 of ASCE 4-16) where the axial rigidity of the shear walls is taken as EcAc for both cracked and uncracked conditions.

Analyses Involving Accident Thermal Conditions

Section N9.2.4 describes analysis requirements for SC walls which include performing a heat transfer analysis and then using the results to develop thermal loading for the SC wall sections. This is the common approach for evaluating structures for thermal loading effects. In addition, this section of N690-18 specifies that the required out-of-plane flexural strengths in the SC wall for interior regions (away from connection regions) caused by thermal gradients shall not exceed the value given by Equation A-N9-15. This equation is explained in the corresponding Commentary section and more fully developed in the paper by Varma et al., (2009). That paper utilized an analytical investigation using a fiber model for the SC section which compared very well to experimental results. Thus, this approach is considered to be acceptable.

Determination of Required Strengths

The required strengths for SC wall sections in terms of in-plane membrane forces, out-of-plane moments, and out-of-plane shear forces are to be determined using an elastic finite element analysis as described in Section N9.2.5. The six required strengths to be determined for each panel section of SC walls are:

- Required out-of-plane flexure in x direction
- Required out-of-plane flexure in y direction
- Required twisting moment
- Required membrane axial strength in x direction
- Required membrane axial strength in y direction
- Required membrane in-plane shear strength

Section N9.2.5 specifies that the required strength for each member load type may be determined by averaging the demand over areal extents of the wall (referred to as "panel sections") that are less than or equal to twice the wall thickness in length and width, except at connections and openings where the panel section dimensions are limited to the wall thickness. These averaging guidelines are generic and may not be suitable in all cases. The implementation of these guidelines or any alternate averaging methodology should be subject to a case-specific review.

For the case of openings and penetrations, and in connection regions, Section N9.2.5 indicates that averaging the demand over panel sections shall be no larger than the section thickness in length and width. The averaging of the demand over a distance equal to one SC wall thickness, around openings and penetrations is judged to be acceptable. For openings, additional design and detailing requirements are specified in Section N9.1.7.

4.24.3 Section N9.3 Design of SC Walls

Uniaxial Tensile Strength

Section N9.3 indicates that the tensile strength contribution from the concrete infill and any steel ribs that may be part of the configuration shall be ignored. As a result, only the steel faceplates are used, and therefore, Section N9.3.1 indicates the uniaxial tensile strength shall be determined in accordance with AISC 360 Chapter D. It is appropriate to neglect the contribution from the concrete because concrete has a low tensile strength and cracks can easily develop in the SC walls. Neglecting any steel ribs is also appropriate because they are generally provided for stiffening effects during fabrication, transportation, and erection, and their cross-sectional area would be quite small relative to the faceplates.

An additional requirement specified in N9.3.1 is that where holes are present in the faceplates, the available rupture strength shall be greater than the available yield strength. This is more conservative than AISC 360 Chapter D, by avoiding tensile rupture in the faceplate.

Compressive Strength

Section N9.3.2 provides requirements for determining the available compressive strength in accordance with AISC 360 Section I2.1b, whereby the faceplates replace the steel shape. It is noted that Section I2.2b is applicable to filled composite members (judged to be comparable to SC wall sections) while Section I2.1b is applicable to encased composite members (i.e., steel members surrounded by concrete). However, Section I2.2b refers to Section I2.1b with several modifications depending if the section is compact, noncompact or slender. These modifications would not apply for SC walls, because Section N9.3.2 redefines the needed modifications to be used with AISC 360 Section I2.1b.

Based on the information provided in N690-18 Commentary Section N9.3.2, it is noted that Equation A-N91-6, provided for calculating compressive strength of SC walls, becomes slightly unconservative for elevated temperatures. Therefore, Equation A-N9-16, should be used for surface temperatures up to 300°F (150°C), consistent with the recommendation made in Section N9.3.2 of the Commentary. For temperatures above this threshold, the basis for the analytical approach should be demonstrated.

Out-of-Plane Flexural Strength

Section N9.3.3 provides the requirements for determining the design flexural strength of SC wall sections. Equation (A-N9-19) is presented to calculate the nominal design flexural strength in terms of the area of steel of the faceplates, yield stress of the faceplates, and SC section thickness. Based on a paper by Sener (2015), the design code in Japan, JEAC-4618 provides an equation for calculating the out-of-plane flexural strength of SC walls. This equation is the same expression as Equation (A-N9-19).

Section N9.3.3 also presents a User Note which indicates that “The nominal flexural strength per unit width, M_n, can also be calculated using reinforced concrete principles (refer to ACI 349 or ACI 349M, Section 10.2).” Section N9.3.5 of the N690-18 Commentary presents the equation that results from following the principles in ACI 349, which could be used (Equation (C-A-N9-11)). The paper by Sener (2015) discusses this equation as well and presents analytical results of testing performed in the US, Japan, South Korea, and China using the simplified Equation (A-N9-19) and Equation (C-A-N9-19) normalized. The calculated results for the specimens show that using both equations results in the calculated nominal strengths greater than the strengths obtained from the test specimen capacities in most cases. These results demonstrate that both Equations (A-N9-19 and CA-N9-11) are acceptable.

In-Plane Shear Strength

Section N9.3.4 provides the requirement for determining the design in-plane strength of SC wall sections. Equation (A-N9-20) is presented to calculate the nominal in-plane shear strength in terms of only the gross area of the faceplates, yield stress of the faceplates and a parameter which is based on the strength-adjusted reinforcement ratio.

The fundamental in-plane behavior mechanics-based model for SC walls was developed by Ozaki et al., (2004), and by Varma et al., (2009). Using the in-plane mechanics-based model, Varma et al, (2014) compared the analytical in-plane shear strengths to the test specimen capacities. The results show that the experimental in-plane strengths are greater than the calculated strengths.

As described in the Commentary and Varma et al. (2011b), the in-plane shear strength can be estimated at the onset point of the yield shear strength of the section. This is given by Equation (C-A-N9-13) which was calibrated to the simplified Equation (A-N9-20).

Since the in-plane shear strength is based on analytical solutions which were verified by experimental results, Equation (A-N9-20) is acceptable.

Out-of-Plane Shear Strength

Section N9.3.5 provides the requirement for determining the nominal out-of-plane shear strength of SC wall sections. The provisions allow establishing the nominal out-of-plane shear strength directly from tests or using the provisions of Section N9.3.4.

If the shear reinforcement spacing is no greater than half of the section thickness, then Equation A-N9-21 is presented to calculate nominal out-of-plane shear strength which is the sum of the shear strength contributed by concrete and shear strength contributed by the shear reinforcement. Additional equations are given on how to calculate the individual concrete and steel reinforcement contributions to shear strength.

If the shear reinforcement is spaced greater than half the section thickness, then the nominal out-of-plane shear strength is the greater of the shear strength of the concrete and the shear strength of the steel. The provisions indicate what equations should be used for this case.

Experimental tests were performed for out-of-plane shear loading and results compared with the ACI 349-06 code equations for determining the shear strength based on the contribution of the concrete and steel. Varma et al., (2011c) concluded from the tests that the ACI code equations for reinforced concrete beams can estimate the out-of-plane shear strength of the SC sections tested. Sener et al., (2015) also tested SC sections to determine the effects of including axial tensile loads simultaneously with out-of-plane shear loads. The conclusion was that even in the presence of significant axial tension, the out-of-plane shear strength could be calculated conservatively using the ACI 349 code provisions for reinforced beams without considering the effects of axial tension.

Section N9.3.5 provides the requirement for determining the nominal out-of-plane shear strength of SC wall sections. It is noted that in Equation A-N9-23, for calculation of the out-of-plane shear strength, the tensile strength, F_t, of the shear reinforcement is used while in ACI 349 the comparable equation uses the yield strength. There is a concern that using Equation A-N9-23 could result in using the rupture (ultimate) tensile stress times the area of the shear reinforcement. However, Chapter D of AISC 360 indicates that the tensile strength of steel members shall be the lower value obtained according the limit states of tensile yielding and tensile rupture. Both are checked because different strength reduction factors are applicable to tensile yielding and tensile rupture, and where applicable (in other situations), tensile yielding uses the gross area and tensile rupture uses the net section. Thus, both cases of tensile yielding and tensile rupture need to be determined for the governing case. Therefore, it should be clarified that the term tensile strength, F_t, to be used in Equation A-N9-23 is to be determined in accordance with Chapter D of AISC 360.

Since the equations used to calculate the out-of-plane shear strengths for SC walls are based on the use of ACI 349 code equations, which were verified by tests to be applicable to SC walls type sections, the approach in Section N9.3.5 is acceptable, provided the clarification described above for tensile strength of the shear reinforcement is addressed.

Strength Under Combined Forces

Section 9.3.6 provides requirements for designing SC walls for combined forces. Section N9.3.6a presents the equation for combined forces due to out-of-plane shear forces and interfacial shear forces. The out-of-plane shear strength of the SC walls is the sum of the strengths from the concrete and the steel reinforcement. Equation A-N9-24 provides an interaction equation for the design of the reinforcement. The ratios of the demand load carried by the reinforcement to the available strength of the reinforcement in the x and y directions (each side of the element) are summed. Then the ratio of the interfacial shear load demand for the reinforcement to the available strength is determined. These two quantities are combined using a 5/3 power interaction equation which needs to be less than or equal to 1.0.

The out-of-plane shear forces in the x and y directions subject the shear reinforcement to axial tension in the same way that occurs for reinforced concrete shear reinforcement. Thus, it is considered acceptable to add the two ratios, corresponding to each direction, linearly. Then the use of a 5/3 power interaction equation, to combine the resulting out-of-plane shear force ratio with the interfacial shear force ratio, is acceptable because the shear reinforcement acts in tension to carry the out-of-plane shear forces, while the shear reinforcement acts in shear to carry the interfacial shear forces. The use of the 5/3 power interaction can be compared favorably with Equation C-J3-5a in AISC 360-16, where a square interaction equation for shear and tension for design of bolts is used. As indicated in AISC 360-16, Equation C-J3-5a is based on “tests that have shown that the strength of bearing fasteners subjected to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse.” The use of the 5/3 power interaction is slightly more conservative than the square interaction equation.

Section N9.3.6b provides the requirements for designing SC walls for the combined three in-plane forces (axial membrane forces in the x and y directions and in-plane shear force) and three out-of-plane moments (two flexural moments and a twisting moment). The approach developed utilizes interaction equations in terms of the maximum and minimum required principal in-plane strengths. As described in the Commentary and Varma et al., (2014) a simplified interaction surface in principal force space for checking the design adequacy of the SC panel section was developed. Varma et al., (2014) verified the conservatism of this approach by developing a mechanics-based model that accounts for the combined in-plane forces and moments. In addition, a nonlinear inelastic finite element model of the SC panel section subjected to the combined in-plane forces and moments was developed. A comparison of the design approach in Section N9.3.6b, mechanics-based model, and the nonlinear finite element model was performed. The results show that the design approach is conservative when compared to the mechanics-based model and the finite element model. Thus, the design equations are considered to be acceptable.

Section 9.3.6b also provides an alternate form of the interaction equations which are in terms of the required in-plane membrane strengths. The alternate interaction equations were confirmed algebraically to be equivalent to the interaction equations. Thus, these set of equations are also considered to be acceptable.

4.24.4 Section N9.4 Design of SC Wall Connections

Section N9.4 provides design requirements for various types of connections which consist of the following:

- Splices between SC wall sections
- Splices between SC wall and reinforced concrete (RC) wall sections
- Connections at intersections of SC walls
- Connections at the intersection of SC with RC walls
- Anchorage of SC walls to RC basemats
- Connection of SC walls to RC slabs

General Provisions

General provisions for design of connections is presented in Section N9.4.1. They describe various connectors consisting of steel headed stud anchors, anchor rods, tie bars, reinforcing bars and dowels, post-tensioning bars, shear lugs, embedded steel shapes, welds, and bolts, rebar mechanical couplers, and direct bearing in compression. The provisions warn that the direct bond transfer between the faceplate and concrete shall not be considered as a valid connector or force transfer mechanism.

All of these provisions are considered to be acceptable because if properly designed, the various types of steel connectors identified above can adequately transfer forces between the adjacent structural members. To provide further guidance on design of SC walls and connections, a User Note is presented in Section N9.4.1 which refers to AISC Design Guide 32, Design of Modular Steel-Plate Composite Walls for Safety-Related Nuclear Facilities. It should be noted that no review has been performed of this design guide since it is a guide, not part of the N690 Specification and it’s beyond the scope of the review for this project.

Required Strength

The only provision specifically given for required strength is presented in Section N9.4.2. The required strength for connections must be either

- 125% of the smaller of the corresponding nominal strengths of the connected parts, or
- 200% of the required strength due to seismic plus 100% of the required strength due to nonseismic loads (including thermal loads)

The use of a load increase of 1.25 for the first approach is considered acceptable because it is intended to ensure that connections are stronger than the connected parts. The use of the 1.25 load factor is consistent with ACI 349-13 which uses this factor to account for strain hardening due to rotation at a joint and overstrength that would be expected for the SC walls. As indicated in the User Note, the first approach is preferred. However, in situations where it is not practical (e.g., SC wall connected to reinforced concrete section, both having very significant nominal strengths substantially beyond the demand loads), it is unreasonable to design the connection to be 1.25 times the smaller of the strengths of the connected parts. In this case, the second approach using 2.0 times the required strength due to seismic plus 100% of the required strength due to nonseismic loads (including thermal loads) is reasonable.

Available Strength

Section N9.4-3 provides the requirements for determining the available strength of connections. The available strength is calculated from the available strength of the connectors that contribute to the force transfer mechanism. The available strength of the connectors is determined based on other Chapters in N690-18 or ACI 349 depending on the type of connector used, as described below.

a. For steel headed stud anchors -Specification Section I8.3.
b. For welds and bolts - Specification Chapter J.
c. For compression transfer via direct bearing on concrete -Specification Section I6.3a.
d. For shear friction load transfer mechanism - ACI 349 or ACI 349M, Section 11.7.
e. For embedded shear lugs and shapes - ACI 349 or ACI 349M, Appendix D.
f. For anchor rods - ACI 349 or ACI 349M, Appendix D.

For the first three connectors listed above, the requirements for determining the available strength refer to other Chapters in the N690-18 which specifically address these types of connectors. These other Chapters were reviewed and evaluated elsewhere in this report. For the last three connectors listed above, the requirements refer to ACI 349 which address the calculation of available strength for steel elements embedded in concrete, and thus, these provisions are also acceptable. It is noted that for Item d, reference is made to Section 11.7 of ACI 349 or ACI 349M. Section 11.7 for shear friction is appropriate for editions prior to and including ACI 349-06. Thereafter, the reference should be to Section 11.6 of ACI 349-13 for shear friction.

4.24.5 Design of Attachments to SC Walls

N690 does not provide requirements or guidance for analysis and design of attachments to the SC walls. While there is a wide variety of attachments that could be made to the SC walls, some provisions or guidance would be useful. These would consist of attachments intended to withstand large loads (e.g., snubbers supporting a steam generator), typically known ahead of time, or attachments intended to withstand smaller loads (e.g., small diameter piping and conduit (often field run) and light equipment).

For large loads, the locations should be known when designing the SC walls, and thus, the design of the SC walls should incorporate features that can absorb and distribute these loads adequately. In this case, the design would be expected to include potentially some combination of the following: a thickened faceplate at the attachment location, through bolts, embedded plates between the faceplates, and embedded reinforcement. When performing the analysis of the SC structure, the additional attachment loads could be applied to the model along with the other loads imparted on the structure, assuming it is appropriate to decouple the attached component(s) that impose the load on the SC walls. Decoupling criteria for seismic loading are provided in SRP Section 3.7.2.

For smaller loads, often the locations will not be known ahead of time, and so, it would be prudent to incorporate in the analysis and design of the SC structure some estimated uniform load. This should be included in the overall structural model to properly capture the mass of the components. Then, when designing the localized attachments to the faceplate, the welds to the attachment and the SC wall faceplate should be evaluated.

Also, an evaluation should be made for the effects of temperature on the concrete due to welding of attachments to the faceplate after the concrete is cured.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Currently, guidance for design of safety-related steel structures for nuclear power plants is presented in Sections 3.8.3, 3.8.4, and 3.8.5 of NRC NUREG-0800, Standard Review Plan (SRP). These sections of the SRP endorse the ANSI/AISC N690-1994 (R2004) Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities. Since the publication of N690-1994 (R2004), the specification has been revised several times, with the most current published version being N690-18, dated June 2018.

N690-18 differs significantly from N690-1994 (R2004) reflecting advances and improvements in steel design gained from experience, analytical studies, and experimental data. With the development of new designs for nuclear power plants, there is also a need to update the NRC guidance to reflect recent editions of design codes, such as the N690 Specification for steel structures. In addition, new nuclear power plant designs are expected to utilize SC type structures. Starting with N690s1-15, provisions for design of SC walls were introduced.

Therefore, the NRC identified a need to review N690-18 and the referenced AISC 360-16 for application to nuclear power plant steel structures. BNL conducted research on behalf of the NRC to review the updated N690-18 and the referenced AISC 360-16 specifications, including assessment of the new SC design provisions. The results of this effort provide a technical basis for the NRC to develop regulatory guidance for the design of safety-related steel structures and SC walls.

It has been concluded that the current version of N690-18, and the referenced AISC 360-16 specification, are a significant enhancement from the N690-1994 (R2004) edition. The current specifications contain substantial upgrades in the design of structural steel and SC wall sections.

If N690-18 is endorsed by the NRC (with applicable regulatory positions), it will allow for clearer and more consistent regulatory guidance, which would benefit the NRC staff and the commercial nuclear power industry. Although BNL recommends that most of N690-18 can be accepted, Section 5.2 includes several recommendations for qualification, exception, or addition, for consideration by the staff in developing their regulatory guidance.

5.2 Summary of Issues and Recommendations

The review of the N690-18 and referenced AISC 360-16 specifications has concluded that the specifications can be endorsed by the NRC for the design of safety-related steel structures and SC walls at nuclear power plants, provided certain qualifications are included in the regulatory guidance. This section of the report summarizes the issues that were identified and presents recommendations to address the issues. Further detailed information on the issues and recommendations are presented in Section 4 of this report. All provisions of N690-18 and AISC 360-16, with the exceptions described below, are considered to be acceptable.

1. Section NA1 specifically excludes pressure retaining components (e.g., pressure vessels, valves, pumps and piping) from the scope of N690-18. All types of supports for these components within the jurisdictional boundary of the ASME Code should also be excluded.

These supports are addressed by the ASME Boiler and Pressure Vessel Code, Section III, Subsection NF.

For Rolled Heavy Shapes and Built-Up Heavy Shapes, subsections NA3.1c and NA3.1d in ANSI/AISC N690-18 require that the design documents identify welded connections that are determined by the engineer of record to be susceptible to lamellar tearing. Those two provisions also require that a plan shall be developed to mitigate the conditions creating the potential for lamellar tearing. To be consistent with N690-94 (R2004), Section Q1.4.1, this plan should include the ultrasonic examination or testing requirements in NA3.1c and NA3.1d of N690-12s1, unless otherwise justified.

2. The methods for design and construction related to QA/QC found in RG 1.28 should be followed.

3. N690-18 Section NB2, Loads and Load Combinations, replaces AISC 360-16 Section B2 in its entirety. This was necessary because AISC 360-16 Section B2 does not identify the specific loads and load combinations. It only indicates that the loads and load combinations shall be those stipulated by the applicable building code, and in the absence of a building code it references ASCE/SEI 7.

   Section NB2 is divided into subsections that define normal loads (NL), severe environmental loads (SEL), extreme environmental loads (EEL) and abnormal loads (AL). Then, it defines the load combinations for the LRFD approach and for the ASD approach. According to the N690-18 Commentary, the pertinent load combinations in Section NB2 come from ACI 349 (2013), SRP 3.8.3 and 3.8.4 (2013) and RG 1.142 (2001).

   The LRFD load combinations did not exist in N690-1994 (R2004) and so they had not been reviewed and endorsed previously by the NRC for design of nuclear safety-related steel structures. The ASD load combination approach has some notable differences from the allowable stress method presented in N690-1994 (R2004). These differences are due to advances in the development of the N690 specification over the years and the principle of action-companion action load combination approach. The action-companion action load combination approach considers that when a primary load in the load combination peaks the other companion loads would not reach their peak value at the same time.

   Based on the evaluation of the N690-18 LRFD and ASD approaches described in Section 4 of this report, the proposed revisions to the load combinations are presented below in black font.

Load and Resistance Factor Design (LRFD)

Proposed Guidance for DG LRFD	Load, Eqn. #
U = 1.4(D + F) + (**1.0**R0 + To) + 1.0C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + **1.6**Ro + 1.2To) + 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + **0.8**Ro + 1.2To) + 1.4C + 1.6(Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.0W + 1.0C + 0.5(Lr or S or R)	SEL, (NB 2-4)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.6Eo + 1.0C + 0.2(Lr or S or R)	SEL, (NB 2-5)
U = D + F + **1.0**L + H + To + Ro + Es + C	EEL, (NB2-6)
U = D + F + **1.0**L + H + To + Ro + Wt	EEL, (NB2-7)
U = D + F + **1.0**L + H + Ta + Ra + 1.4Pa + C	AL, (NB2-8)
U = D + F + **1.0**L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es	AL, (NB2-9)

*Footnotes and caveats applicable to this table are given in the Attachment to this report.

Allowable Strength Design (ASD)

Proposed Guidance for DG ASD	Load, Eqn. #
U = (D + F) + (Ro + To) + C	NL, (NB2-10)
U = (D + F) + (**L** + H + Ro + To) + C + **0.75**(Lr or S or R)	NL, (NB2-11)
U = (D + F) + (0.75L + 0.75H + **0.75Ro** + To) + C + **1.0**(Lr or S or R)	NL, (NB2-12)
U = (D + F) + (**1.0**L + **1.0**H + Ro + To) + 0.6W + C + 0.75(Lr or S or R)	SEL, (NB2-13)
U = (D + F) + (**1.0**L + **1.0**H + Ro + To) + Eo + C + 0.75(Lr or S or R)	SEL, (NB2-14)
U = D + F + L + H + To + Ro + Es + C	EEL, (NB2-15)*
U = D + F + L + H + To + Ro + Wt	EEL, (NB2-16)*
U = D + F + L + H + Ta + Ra + Pa + C	AL, (NB2-17)*
U = D + F + L + H + Ta + Ra + Pa + Yr + Yj +Ym + 0.7Es	AL, (NB2-18)*

*Footnotes and caveats applicable to this table are given in the Attachment to this report.

4. For Chapter NC - Design for Stability, the Staff accepts the referenced AISC 360-16 Chapter C, subject to the following qualifications:

    a) the provisions of Appendix 7 for the Equivalent Length Method (ELM) should only be implemented for safety-related steel structures in nuclear facilities in cases where any restrictions on its use are clearly satisfied and minimal judgement is required to determine K (effective length factor);

    b) the First Order Analysis Method (FOM), using B_1 and B_2 factors from Appendix 8 to simulate second order effects, should only be implemented for safety-related steel structures in nuclear facilities in cases where any restrictions on its use are clearly satisfied;

    c) Prediction of elastic stability using the Direct Second Order Analysis Method (DM) is acceptable. Since it only considers geometric nonlinearities and initial imperfections, the stress state at the onset of instability should be confirmed to be in the elastic range to ensure valid results.

The DM, considering both geometric and material nonlinearities, and accounting for initial imperfections, will provide the most accurate predictions of structural collapse due to instability or excessive deformation. When accurate displacement predictions are needed to satisfy a maximum displacement criterion that exceeds the elastic response limit (e.g., to preclude structure-to-structure interaction), this is the only viable analysis method.

5. Several chapters in N690-18 and the referenced AISC 360-16 refer to requirements in ACI 318 which are not consistent with current NRC regulatory guidance. In accordance with RG 1.142 Rev. 3, ACI 349-13 should be used along with the regulatory positions in RG 1.142 Rev. 3 rather than ACI 318, unless otherwise justified.

6. Consistent with RG 1.142 Rev. 3, high-strength reinforcement (yield strength greater than 60,000 ksi) should not be used in design, unless otherwise justified. If high-strength reinforcement is used, applicants should demonstrate their adequacy for specific use in the design by testing, analysis, or performance evaluation.

7. Chapter NI incorporates AISC 360-16 Chapter I in its entirety for Design of Composite Members. This chapter addresses composite members composed of rolled or built-up structural steel shapes or HSS and structural concrete acting together, and steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending.

   For the first time, AISC 360-16 incorporated provisions for applying the direct analysis method to composite members, in a new Paragraph I1.5 “Stiffness for Calculation of Required Strengths”, and the accompanying Commentary. The User Note in Paragraph I1.5 references the Commentary for “Stiffness values appropriate for the calculation of deflections and for use with the effective length method …” The accompanying Commentary identifies caveats when using the provisions of Paragraph I1.5. It states that “Research indicates that the stiffness prescribed in this section may result in unconservative errors for very stability sensitive structures (Denavit et al., 2016a)”. Therefore, the current design provisions of AISC 360-16, Paragraph I1.5, for applying the direct analysis method to composite members, are acceptable if the findings of Denavit et al., 2016a are addressed and demonstrated to be not applicable.

   A second caveat in the Commentary states: “The Specification has traditionally not accounted for long-term effects due to creep and shrinkage; as such, the stiffness prescribed in this section was developed based on studies examining only short-term behavior. Refer to Commentary Sections I1 and I3.2 for additional discussion.” Therefore, in implementing the provisions of AISC 360-16 Chapter I “Design of Composite Members”, the effects of concrete shrinkage and creep on the structural stiffness and strength of composite members should be considered and, as necessary, incorporated in the design process.

8. N690-18 Chapter NG - Design of Members for Shear, accepts in its entirety Chapter G of AISC 360-16. Section G2.1, of the Commentary to AISC 360-16 states: “The provisions of Section G2.1 assume monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).”

The design basis for commercial nuclear power plant safety-related steel structures should not permit “load reversals causing cyclic yielding over large portions of a web.” Consequently, the provisions of Section G2.1 are acceptable for design of commercial nuclear power plant safety-related steel structures.

9. N690-18 Appendix N1, Design by Advanced Analysis, incorporates AISC 360-16 Appendix 1, with certain modifications. The following qualifications apply to Appendix N1:

    a) Nonlinear Inelastic Analysis, per N690-18 Section NB3.14, is acceptable for treatment of impactive and impulsive loads.

    b) Nonlinear Inelastic Analysis, per N690-18 Appendix N1, Section 1.3.1 supplement to AISC 360-16 Appendix 1, Section 1.3.1, is acceptable for treatment of thermally induced local inelastic effects.

    c) Nonlinear Inelastic Analysis, per AISC 360-16 Appendix 1, Section 1.3, is acceptable for structural evaluation of Seismic Category II (non-safety related) steel structures to address Seismic II/I interactions. The details of each analysis are reviewed by the staff.

    d) Nonlinear Elastic Analysis, per AISC 360-16 Appendix 1, Section 1.2, is acceptable for use with AISC 360-16 Chapter C for elastic stability analysis of safety-related steel building structures, in cases where stresses are below the elastic limit at the onset of instability.

    e) Nonlinear Inelastic Analysis, per AISC 360-16 Appendix 1, Section 1.3 is acceptable for use with AISC 360-16 Chapter C for inelastic stability analysis of safety-related steel building structures, in cases where stresses exceed the elastic limit prior to instability.

10. N690-18 Appendix N3, Design for Fatigue, incorporates by reference AISC 360-16 Appendix 3 in its entirety. While there are significant changes from the comparable material in N690-94 (2004 supplement) Appendix QB, the use of AISC 360-16 Appendix 3 is acceptable for safety-related structures in nuclear facilities. Relaxation of the allowable stress range vs. number of loading cycles, compared to N690-1994 (2004 supplement) Appendix QB, is acceptable, based on the referenced test results.
    Also, AISC 360-16 Appendix 3, Section 3.1 defines an upper limit on the allowable stress range that may supersede the allowable stress range obtained using Eqn. (A-3-1). The analyst must check both and use the lower of the two allowables.

11. N690-18, Appendix N4 addresses structural design for fire conditions. Section N4.1 indicates that the Appendix does not address either “Important to Safety” structural steel members or loading conditions associated with a facility fire. Since this Appendix does not apply to Important to Safety structural steel members, an exception to Appendix N4 should be taken.

12. Appendix N5, entitled Evaluation of Existing Structures, does not address seismic and other dynamic loads. Therefore, an exception to Appendix N5 should be taken.

13. Appendix N9 Steel-Plate Composite (SC) Walls:

    N690-18 Appendix N9 provides updated criteria for the design of SC walls in safety-related nuclear facilities. SC walls are considered modular type members constructed from steel faceplates spaced apart, connected with steel ties, and then filled with concrete. The faceplates also utilize steel anchors welded to the plates which, along with the ties, make the entire section act monolithically. Since there are no comparable provisions in AISC 360 Specification, Appendix N9 provides a complete set of provisions without reference to AISC 360.

    (a) <u>Design for Impactive and Impulsive Loads</u>

    N690-18 Section N9.1.6, Design for Impactive and Impulsive Loads, and its subsections identify the criteria for design of SC walls for impactive and impulsive loads. As explained in the Commentary to Section N9.1.6, the section is based on ACI 349-13, Appendix F - Special Provisions for Impulsive and Impactive Effects. While the criteria and provisions in N690-18 Section N9.1.6 are specific to SC walls, some of the criteria in this section of N690-18 are not consistent with those in ACI 349-13 Appendix F and NRC Regulatory Guide 1.142 Rev. 3 for reinforced concrete members. Based on the evaluation in Section 4 of this report, the provisions in N690-18 Section N9.1.6 are considered acceptable with the additions and exceptions identified below.

    1. Impactive and impulsive loads are assumed to be concurrent with other loads (e.g., dead and live loads) in determining the required strength of structural elements.

    2. For impulsive loads, the strength available for impulsive loads is at least 20 percent greater than the magnitude of any portion of the impulsive loading, which is approximately constant for a time equal to or greater than the first fundamental period of the structural member. See Table 6 in the Attachment for further details.

    3. In addition to the deformation limits under items 4. to 7. below, the maximum deformation shall not result in the loss of intended function of the structural wall nor impair the safety-related function of other systems and components.

    4. For flexure-controlled SC walls as defined in Section N9.6b of ANSI/AISC N690-18, the permissible displacement ductility ratio demand should satisfy all of the following:

        - ductility ratio less than or equal to 10,
        - principal strain of the faceplates less than or equal to 0.05 (Johnson et al., 2014), and
        - rotational capacity of any yield hinge less than or equal to 0.07 radians (4 degrees) [Bruhl et al., 2017).

    5. For SC walls resisting axial compression, the permissible displacement ductility ratio should be as shown in 5.1 to 5.3.

5.1 When compression controls the design as defined by the balanced point in a load-moment interaction diagram, the permissible ductility ratio shall be 1.0.

5.2 When the compression load does not exceed $0.1(f'_c A_g)$, where A_g is the sum of the area of concrete infill and the net area of the faceplates, or one-third of that which would produce balanced strain conditions, whichever is smaller, the permissible ductility ratio should be as given in 4.

5.3 The ductility ratio varies linearly between 1.0 and that given in 4. for conditions between those described in 5.1 and 5.2.

6. The permissible displacement ductility ratio in flexure should not exceed 3.0 for loads such as blast and compartment pressurization, which could affect the integrity of the structure as a whole.

7. For shear-controlled SC walls, with yielding reinforcement spaced at section thickness divided by two or smaller, the ductility ratio is limited to 1.3. For shear-controlled SC walls with other configurations of yielding or nonyielding reinforcement, the ductility ratio is limited to 1.0.

8. The shear strength under local loads considers reaction shear at the supports and punching shear adjacent to the load.

8.1 Local loads may be impulsive or impactive, except that for impactive loads, satisfaction of criteria for perforation shall be used in place of punching shear requirements.

8.2 The shear strength is determined in accordance with the provisions in N690 Section N9.3, Design of SC Walls, using the appropriate dynamic increase factors (DIFs) in Table A-N9.1.1 for the required concrete and steel material properties.

8.3 In the case of the reaction shear (beam action condition) at the supports, the effective width of the critical section for the shear beam capacity at the supports is to be determined according to the zone of influence induced by the local loads instead of the entire width of the support. The zone of influence induced by the concentrated loads may be determined, for example, by an analysis [see Lantsoght, Eva O. L., (2012)].

9. Design of SC walls or SC structural wall systems for impactive loads satisfies the criteria for local effects and overall structural response. Their structural response is determined by the methods for impulsive loads in Section N.9.1.6c. Local effects include penetration, perforation and punching shear. The penetration depth and required concrete and faceplate thickness required to prevent penetration are from applicable rational methods or pertinent test data.

10. Evaluation of loads from malevolent, beyond-design-basis, aircraft impacts are in the scope of RG 1.127 and outside the scope of this report.

(b) Determination of Required Strengths

N690-18 Section N9.2.5 specifies that the required strength for each member load type may be determined by averaging the demand over areal extents of the wall (referred to as “panel sections”) that are less than or equal to twice the wall thickness in length and width, except at connections and openings where the panel section dimensions are limited to the wall thickness. These averaging guidelines are generic and may not be suitable in all cases. The implementation of these guidelines or any alternate averaging methodology should be subject to a case-specific review.

(c) Compressive Strength

Based on the information provided in N690-18 Commentary Section N9.3.2, it is noted that Equation A-N91-6, provided for calculating compressive strength of SC walls, becomes slightly unconservative for elevated temperatures. Therefore, Equation A-N9-16, should be used for surface temperatures up to 300°F (150°C), consistent with the recommendation made in Section N9.3.2 of the Commentary. For temperatures above this threshold, the basis for the analytical approach should be demonstrated.

(d) Out-of-Plane Shear Strength

N690-18 Section N9.3.5 provides the requirement for determining the nominal out-of-plane shear strength of SC wall sections. It is noted that in Equation A-N9-23, for calculation of the out-of-plane shear strength, the tensile strength, F_t, of the shear reinforcement is used while in ACI 349 the comparable equation uses the yield strength. There is a concern that using Equation A-N9-23 could result in using the rupture (ultimate) tensile stress times the area of the shear reinforcement. However, Chapter D of AISC 360 indicates that the tensile strength of steel members shall be the lower value obtained according the limit states of tensile yielding and tensile rupture. Both are checked because different strength reduction factors are applicable to tensile yielding and tensile rupture, and where applicable (in other situations), tensile yielding uses the gross area and tensile rupture uses the net section. Thus, both cases of tensile yielding and tensile rupture need to be determined for the governing case. Therefore, it should be clarified that the term tensile strength, F_t, to be used in Equation A-N9-23 is to be determined in accordance with Chapter D of AISC 360.

(e) Design of Attachments to SC Walls

N690 does not provide requirements or guidance for analysis and design of attachments to the SC walls. While there is a wide variety of attachments that could be made to the SC walls, some provisions or guidance would be useful. These would consist of attachments intended to withstand large loads (e.g., snubbers supporting a steam generator), typically known ahead of time, or attachments intended to withstand smaller loads [e.g., small diameter piping and conduit (often field run) and light equipment].

For large loads, the locations should be known when designing the SC walls, and thus, the design of the SC walls should incorporate features that can absorb and distribute these loads adequately. For smaller loads, often the locations will not be known ahead of time, and so, it would be prudent to incorporate in the analysis and design of the SC structure some estimated uniform load. This should be included in the overall structural model to properly capture the mass of the components. Then, when designing the localized attachments to the faceplate, the welds to the attachment and the SC wall faceplate should be evaluated.

Also, an evaluation should be made for the effects of temperature on the concrete due to welding of attachments to the faceplate after the concrete is cured.

Editorial Corrections

As a result of the review of N690-18, several items were identified which appear to be editorial type corrections that should be incorporated into an errata or the next version of the specification. These items are described below.

1. N690-18 Table NB3.2 provides, under the last column heading "Example", sketches showing different types of structural elements corresponding to unstiffened and stiffened members. For the information missing in this column for unstiffened elements, the sketches shown in Table NB3.2 of N690-12 should be used.

2. The title for N690-18 Figure C-A-N9.1.21 in Section N9.1.7a of the Commentary to N690-18 should be identified as "Small circular opening – detailing illustration for fully developed edge with flange plate thickness < $1.25t_p$".

3. The definitions for two parameters used in N690-18 Equations A-N9-8 and A-N9-8M in Section N9.2 of N690-18 should be identified as given below.

   - c_2 = calibration constant for determining effective flexural stiffness
   - $\overline{\rho}$ should be replaced with ρ' in the definition for the stiffness-adjusted reinforcement ratio

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ATTACHMENT

Allowable Strength Design (ASD) and Load and Resistance Factor Design (LRFD) Load Combinations and SC Wall Design for Impactive and Impulsive Loads

This Attachment was contributed by USNRC/RES staff (J. Pires, M. Rolon Acevedo, F. Sock) and was reviewed by BNL (J. Braverman, R. Morante) for concurrence. It provides the detailed technical basis for acceptable load combinations and SC Wall design for impactive and /impulsive load design.

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BACKGROUND

Allowable Strength Design Method and Load and Resistance Factors Design Method

Since the publication of the 2005 AISC 360 Specification (herein called the Specification), the same strength limit states are the basis for both the Allowable Strength Design (ASD) method and Load and Resistance Factor Design (LRFD) method [1]. This means that the nominal limit state strength, R_n is the same for the ASD and the LRFD methods. According to [2], the AISC 360 Specification (herein called the Specification) links all strength provisions to the nominal strength of an element and then applies either a safety factor for ASD or a resistance factor for LRFD to determine the available strength. The result of this very simple concept is that there is a direct relationship between the safety factor and the resistance factor for every design consideration. The design equation for the ASD and LRFD are, respectively,

$R_n \geq [\ \sum \gamma_{iA} Q_i\ ]/\Omega$ (ASD)

$R_n \geq \phi[\ \sum \gamma_{iL} Q_i\ ]$ (LRFD)

where

γ_{iA}	=	load factors for the ASD approach (usually 1.0)
Ω	=	ASD factor of safety
γ_{iL}	=	load factors for the LRFD approach
Q_i	=	nominal loads for the ASD and LRFD approach
ϕ	=	resistance factors for the LRFD

According to [1], prior to the 2005 AISC 360 Specification, the term ASD in the 360 Specification referred to the allowable stress design approach. Prior to 2005, the allowable stress design approach was based on the concept that the maximum stress from the applied loads shall not exceed a specified allowable stress. According to [1], the safety factors were primarily based on experience and workmanship and had remained unchanged for about 75 years. A single factor of safety was used for each load combination and the load factors were taken to be equal to 1.0. Reference 1 also states that while the level of safety provided by the traditional allowable stress design had always been variable and unknown, and varied from load combination to load combination and from limit state to limit state, the structures designed using the traditional allowable stress design approach performed satisfactorily.

When developing the LRFD approach, the AISC determined resistance factors for various limit states and components, e.g., tensile yielding limit state of tension members, tensile rupture of tension members, compression members, flexure of beams, shear of beams, welds and fasteners, in conjunction with load factors for prescribed load combinations to achieve desired reliability indices for the load combinations of interest to the Specification.

For the Allowable Strength Design method, which uses the same limit states and same limit state strength as the LRFD, the factors of safety were chosen in order to obtain reliability indices as close as possible as those obtained with the LRFD while using load factors as close as possible to 1.0 for the load combinations.

This approach is illustrated in the following for the dead load and live load combination:

- LRFD
  - $\phi R_n = 1.4D$
  - $\phi R_n = 1.2D + 1.6L$

- ASD
  - $R_n / \Omega = D + L$

Consider L = 3D. In this case, it is

- LRFD
  - $\phi R_n = 1.4D$, or $R_n = 1.4D/\phi$
  - $\phi R_n = 1.2D + 1.6L = 1.2D + 1.6(3)D = 6D$ or $R_n = 6D/\phi$

- ASD
  - $R_n / \Omega = D + L = 4D$, or $R_n = 4D/\Omega$

Equating R_n from the ASD and LRFD method

$R_n = 6D/\phi = 4D/\Omega$, or
$\Omega = 6/(4\phi) = 1.5/\phi$

For a tension member and the yielding limit state, $\phi = 0.9$ and $\Omega = 1.5/0.9 = 1.67$. For the tensile rupture of a tension member $\phi = 0.75$ and $\Omega = 1.5/0.75 = 2.0$.

For the general case of yielding of a tension member and an arbitrary L/D ratio:

- LRFD
  - $0.9 R_n = 1.4D$ or $R_{nLRFD} = 1.56D$
  - $0.9 R_n = 1.2D + 1.6L$ or $R_{nLRFD} = 1.33D + 1.78L$

- ASD
  - $R_n / 1.67 = D + L$, or $R_{nASD} = 1.67D + 1.67L$

The ratio R_{nLRFD} / R_{nASD} is then

- $R_{nLRFD} / R_{nASD} = [0.8+1.07(L/D)]/[1 + (L/D)]$

which is plotted in Figure 1. This ratio is 1.0 for L/D = 3 and 1.03 for L/D = 6, meaning that the ASD approach would result in a nominal strength about 3% less than that obtained using the LRFD. For L<3D, the ratio is 0.98 for L/D = 2 and 0.93 for L/D = 1.0, meaning that the ASD approach can be about 7% more conservative than the LRFD.

Given that for the LRFD case in the Specification, the load combination 1.4D, dead load alone, also must be met, it is $R_{nLRFD} / R_{nASD} = [(1.4/0.9)D]/[1.67D + 1.67L] = (0.93)/[1 + (L/D)]$. This equation produces the same results as the previous bulleted equation for R_{nLRFD} / R_{nASD} given above, when (L/D) = 0.12 and results in $R_{nLRFD} / R_{nASD} = 0.83$.

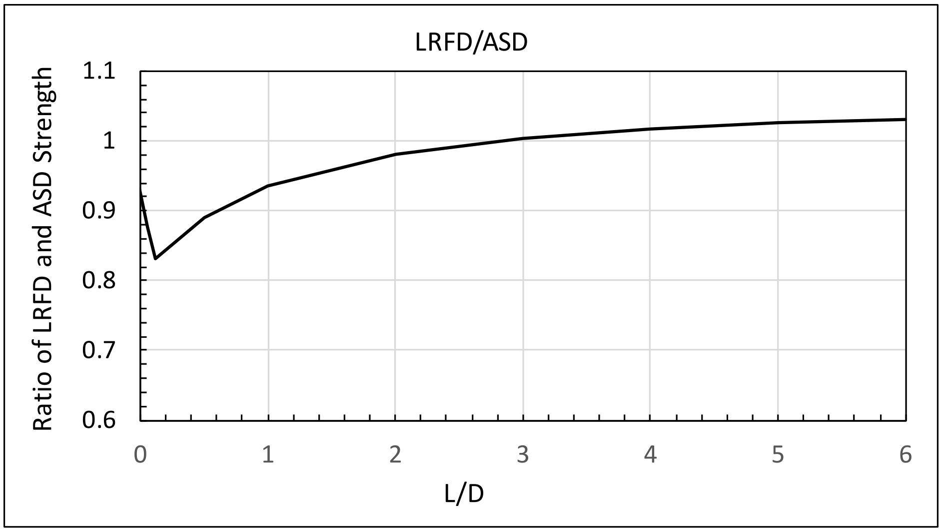


Figure 1. Comparison of LRFD and ASD designs for tension members and live and dead load combination

Load Combinations

The considerations above can also be used to assess the ASD load combinations once LRFD load combinations are chosen. The current AISC approach, when transitioning to the allowable strength design (ASD) method, was to use the same limit states and strength for the ASD and LRFD methods. The load and resistance factors had been established over the previous decades (at first that involved comparisons and calibrations with the allowable stress design method but the LRFD progressively became independent of the allowable stress design method and gained its own experience basis as well). Once the LRFD was established and gained its experience basis it was possible to establish an allowable strength approach using the same limit states as the LRFD and using the load and resistance factors of the LRFD to set the factor of safety and load combinations for the ASD.

The following sections review the load combinations in N690-18 against those in ACI 349-13 for concrete structures and those in RG 1.142 also for concrete structures. Then, they present draft guidance for LRFD load combinations for steel and SC walls based on the N690-18 load combinations and alignment with those in RG 1.142 while accounting for the differences between steel, SC walls and reinforced concrete structures, and new information or insights since the development of RG 1.142. The proposed LRFD load combinations for DG-1304 are then used to develop the draft guidance for the ASD load combinations to obtain a set of consistent load combinations for the ASD and LRFD.

LOAD COMBINATIONS

This section develops the recommendation for the LRFD and ASD load combinations for DG 1304. The process maintains the load combination format in N690-18, which is consistent with that in Revision 3 of Regulatory Guide 1.142. The load factors are consistent with those in Regulatory Guide 1.142 while accounting for differences of the effects of uncertainties of specific loads on steel structures and steel plate composite (SC) walls in contrast to their effects on reinforced concrete (RC) structures.

Comparison of N690-18 and ACI 349-13 Load Combinations

Table 1 provides the LRFD load combinations in N690-18 and ACI-349-13.

Table 1. LRFD load combinations in AISC N690-18 and ACI 349-13

AISC N690-18 Section NB2 LRFD	**Load, Eqn. #**
U = 1.4(D + F) + (1.4Ro + To) + C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.2Ro + 1.2To) + 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + 1.2Ro + 1.2To) + 1.4C + 1.6(Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + W + 1.0C + 0.5(Lr or S or R)	SEL, (NB2-4)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + 1.6Eo + 1.0C + 0.2(Lr or S or R)	SEL, (NB2-5)
U = D + F + 0.8L + H + To + Ro + Es + C	EEL, (NB2-6)
U = D + F + 0.8L + H + To + Ro + Wt	EEL, (NB2-7)
U = D + F + 0.8L + H + Ta + Ra + 1.2Pa + C	AL, (NB2-8)
U = D + F + 0.8L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es	AL, (NB2-9)

ACI 349-13 Section 9.2.1 LRFD*	**Load, Eqn. #**
U = 1.4(D + F) + (1.4Ro + To)	NL, (9-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.2Ro + 1.2To) + 1.4C + 0.5(Lr or S or R)	NL, (9-2)
U = 1.2(D + F) + (0.8L + 0.8H + 1.2Ro + **0.0To**) + 1.4C + 1.6(Lr or S or R)	NL, (9-3)
U = 1.2(D + F) + (**1.6**L + 1.6H + 1.2Ro + **0.0To**) + **1.6**W + **0.0C + 0.0(Lr or S or R)**	SEL, (9-5)
U = 1.2(D + F) + (**1.6**L + 1.6H + 1.2Ro + **0.0To**) + 1.6Eo + **0.0C + 0.0(Lr or S or R)**	SEL, (9-4)
U = D + F + 0.8L + H + T_o + Ro + Es + C	EEL, (9-6)
U = D + F + 0.8L + H + To + Ro + Wt	EEL, (9-7)
U = D + F + 0.8L + H + Ta + Ra + 1.2Pa + C	AL, (9-8)
U = D + F + 0.8L + H + Ta + Ra + Pa + Yr + Yj + Ym + **1.0**Es	AL, (9-9)

[*] The bold type font in this table highlights the differences between the AISC N690-18 LRFD load combinations and the load combinations in Chapter 9 of ACI 349-13.

Differences between the N690-18 and ACI 349-13 are as follows:

- Normal Load (NL) Combinations
  - N690 includes the crane load (C) in the normal load combination for dead loads while ACI does not
  - N690 includes the temperature for operating and shutdown conditions (To) in the normal load combination for roof loads (including snow or rain) while ACI does not
- Severe Environmental Load (SEL) Combinations
  - N690 includes roof loads (including snow and rain)
  - N690 includes the crane loads (C) while ACI does not
  - N690 includes the temperature for operating and shutdown conditions while ACI does not
  - N690 uses a load factor of 1.0 for W while ACI uses a factor of 1.6. The 1.6 in ACI is based on a design wind speed for a 100-year return period recommended in ASCE 7 prior to 2010.
    - The factor of 1.0 for W in N690 is associated with a design wind speed for a 3000-year return period for risk category IV, which is the wind treatment in ASCE 7-16. (The ratio of the nominal wind speeds using the ASCE 7 prior to 2010 and the ASCE

7-16 varies somewhat depending on the site conditions but is about 1.6, which is the difference in the load factors.)

  - N690 uses a load factor of 0.8 for the live load (L) for the wind and operating basis earthquake load (Eo) while ACI 349 uses a factor of 1.6. N690 uses the principle of combining principal actions, in these two cases either W or Eo, with a point in time value for the companion loads like L.

- Extreme Environmental Load (EEL) and Abnormal Load (AL) Combinations
  - The N690 and ACI load combinations are identical except that in NB2-9 N690 uses a load factor of 0.7 for the safe shutdown earthquake (Ess) when combined with accident and impact loads while ACI uses a load factor of 1.0.
    - N690 invokes the principal action/companion action method of combining loads probabilistically. In N690, Ess is the principal action in NB2-6 and its load factor is 1.0 while it is a companion action in NB2-9 where its load factor is 0.7. The probability of simultaneous occurrences of the peak Ess and of the accident and impact loads is small.
    - Even when both are concurrent, the time scales of Ess and the accident/impact loads are different; the peak response to SSE ground motion is reached in a matter of seconds, while the peak response to a LOCA takes longer to reach; in other words, the probability of coincident peak responses is expected to be vanishingly small; combining peak Ess with peak Pa/Ta/Ra is not credible.

Comments on roof loads including snow and rain loads, crane loads and temperature (To):

- Roof loads typically are more important for steel framed buildings than they are for reinforced concrete buildings. Roof loads including snow/rain loads rarely govern the design of roofs consisting of 4-inch thick or more reinforced concrete slabs. However, they can be significant for a steel frame building with a roof system with light-weight concrete on a metal deck.
- Crane and temperature are significant factors in steel-framed industrial-type buildings, and thus crane loads and temperature effects play a larger role in the N690 combinations than they do in ACI 349, thus they are included in the combinations for severe environmental loads NB2-4 and NB2-5.

Comments on hydrostatic fluid loads (F) and earth pressures (H):

- When dead loads stabilize the structure, the load factor should be 0.9 rather than 1.2 (comment 5d(4) in N690 has a strong technical basis and has been substantiated by reliability analysis.
- Fluid pressures (hydrostatic) have been considered as dead loads in chapters 3.8.3 and 3.8.4 of the SRP (NUREG-0800) and should remain as a dead load. Different levels of F loads can be treated as part of different nominal load combinations. Consideration should be given to use F as zero when it acts against destabilizing effects or uplift (commends 5d(3) and 5d(4) in N690).
- The load factor on H, loads due to weight and pressure of soil, water in soil, or other materials, shall be set equal to zero if the structural action due to H counteracts that due to W, Wt, Eo or Ess. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

Extreme Load Wt :

- Design for loads due to accidental explosions, or accidental vehicle impacts, or small aircraft impacts should use load combination (NB9-7) with those loads in lieu of Wt. The Wt load in the load combination (NB9-7) should be evaluated for tornado and for hurricane loads applicable to the site for which related regulatory guidance is in RG 1.76 and RG 1.221, respectively. RG 1.217 addresses effects from beyond-design-basis large aircraft impacts.

Proposed LRFD Load Combinations for the DG endorsing N690-18

RG 1.142 Revision 3 found the load combinations in ACI 349-13 acceptable for the design of safety-related reinforced concrete structures other than reactor vessels and containments with a few exceptions and additions. ACI 349-13 and RG 1.142 use a LRFD approach for design of safety-related concrete structures. Although it may appear desirable to use the same load combinations for reinforced concrete structures and the structures in the scope of N690 (namely steel structures, composite structures and SC walls), uncertainties in the definition of specific loads impact differently the response of structures in the scope of ACI 349-13 and AISC N690-18. In addition, new understanding of load combinations and new definitions of nominal loads should also be accounted for in the DG for the N690 standard.

Table 2. LRFD load combinations in N690-18, RG 142 and proposed for this DG

AISC N690-18 Section NB2	LRFD	Load, Eqn. #
U = 1.4(D + F) + (1.4Ro + To)	+ C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.2Ro + 1.2To)	+ 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + 1.2Ro + 1.2To)	+ 1.4C + 1.6(Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + 1.0W	+ 1.0C + 0.5(Lr or S or R)	SEL, (NB2-4)
U = 1.2(D + F) + (0.8L + 1.6H + 1.2Ro + To) + 1.6Eo	+ 1.0C + 0.2(Lr or S or R)	SEL, (NB2-5)
U = D + F + 0.8L + H + To + Ro + Es	+ C	EEL, (NB2-6)
U = D + F + 0.8L + H + To + Ro + Wt		EEL, (NB2-7)
U = D + F + 0.8L + H + Ta + Ra + 1.2Pa	+ C	AL, (NB2-8)
U = D + F + 0.8L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es		AL, (NB2-9)

RG 1.142	LRFD	Load, Eqn. #
U = 1.4(D + F) + (1.0Ro + To)	(No C load here)	NL, (9-1)
U = 1.2(D + F) + (1.6L + 1.6H + 1.6Ro + 1.6To)	+ 1.4C + 0.5(Lr or S or R)	NL, (9-2)
U = 1.2(D + F) + (0.8L + 0.8H + 0.8Ro + 0.8To)	+ 1.4C + 1.6(Lr or S or R)	NL, (9-3)
U = 1.2(D + F) + (1.6L + 1.6H + 1.6Ro) + 1.6W	(No C or roof loads)	SEL, (9-5)
U = 1.2(D + F) + (1.6L + 1.6H + 1.6Ro) + 1.6Eo	+ 1.4C (No roof loads)	SEL, (9-4)
U = D + F + 1.0L + H + T_o + Ro + Es	+ C	EEL, (9-6)
U = D + F + 1.0L + H + To + Ro + Wt		EEL, (9-7)
U = D + F + 1.0L + H + Ta + Ra + 1.4Pa	+ C	AL, (9-8)
U = D + F + 1.0L + H + Ta + Ra + Pa + Yr + Yj + Ym + 1.0Es		AL, (9-9)

Proposed Guidance for DG LRFD*		**Load, Eqn. #**
U = 1.4(D + F) + (**1.0**R0 + To)	+ 1.0C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + **1.6**Ro + 1.2To)	+ 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + **0.8**Ro + 1.2To)	+ 1.4C + 1.6(Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.0W	+ 1.0C + 0.5(Lr or S or R)	SEL, (NB 2-4)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.6Eo	+ 1.0C + 0.2(Lr or S or R)	SEL, (NB 2-5)
U = D + F + **1.0**L + H + To + Ro + Es	+ C	EEL, (NB2-6)
U = D + F + **1.0**L + H + To + Ro + Wt		EEL, (NB2-7)
U = D + F + **1.0**L + H + Ta + Ra + 1.4Pa	+ C	AL, (NB2-8)
U = D + F + **1.0**L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es**		AL, (NB2-9)

* The bold type font in this table highlights the differences between the AISC N690-18 LRFD load combinations and the recommended LRFD load combinations.

** As stated in commentary NB2.5 of N690-18, when a defined time-phase relationship is lacking, designers have resorted to several approaches to account for the potential interaction of the dynamic loads. One approach, called absolute or linear summation (ABS) method, linearly adds the absolute values of the peak structural response due to the individual dynamic loads. A second approach, called the square root of the sum of the squares (SRSS) method, uses a combined response equal to the square root of the sum of the squares of the peak responses from individual dynamic loads. If using 0.7Es in NB9-9, the SRSS approach is not permitted. If using 1.0Es in NB2-9, then SRSS is permitted.

Differences between this DG and RG 1.142:

- Crane loads (C) and temperature loads (To) are significant factors in steel-framed industrial-type buildings, and thus crane loads and temperature effects play a different role in the N690 combinations than they do in concrete structures. Therefore, C and To loads are included in both severe environmental load combinations, NB2-4 and NB2-5. Crane loads (C) are also included in the normal load combination NB2-1.

- Because of the different role of temperature in operating and shutdown conditions (To) for steel and SC structures as compared to reinforced concrete structures, the load factor for To in the N690-18 is retained for the normal load combinations NB2-2 and NB2-3. The load factor for the crane loads (C) in N690-8 for the severe load environmental load combinations is kept as 1.0 as these are companion loads. A load factor of 1.4 for (C) is used in the normal load combinations NB2-2 and NB2-3.

- Roof loads typically are more important for steel framed buildings than they are for reinforced buildings. Roof loads including snow/rain loads rarely govern the design of the roofs of consisting of 4-inch thick or more reinforced concrete slabs. However, they can be significant for a steel frame building with a roof system with light-weight concrete on a metal deck. Therefore, roof loads, snow loads and rain loads are included in the severe environmental load combinations, load combinations NB2-4 and NB2-5.

Differences between this DG and N690-18

- Uncertainties in the reaction loads (Ro) are greater than those in the dead loads. Unless otherwise justified, the DG treats the reaction loads (Ro) load factors as those for the live loads (L).

- In the severe environmental load combinations, the load factors for the live loads (L) is retained at 1.6 until further justification is provided to reduce it to a lower value to account for the low probability of simultaneous occurrence of Eo with the lifetime maximum live load L. The extreme environmental load combinations NB2-6 and NB2-9 account for the low probability of the simultaneous occurrence of the lifetime maximum live load with the earthquake loads, in this cases Es.

- Unless justified, the load factors for the live loads (L) in the extreme environmental load combinations is 1.0 because although L is a companion load in those combinations, its mean value tends to be close to its nominal value (Regulatory Guide 1.142, Revision 3).

- The load factor for the severe wind loads in severe environmental load combination NB2-4 is 1.0 as opposed to 1.6 in RG 1.142. RG 1.142 retained the load factor of 1.6 for W, which assumed nominal values for W associated with wind speeds for a 100-year return period as recommended in ASCE 7 prior to 2010. The factor of 1.0 for W in N690-18 assumes nominal values for W associated with a design wind speed for a 3000-year return period for risk category IV, which is the wind treatment in ASCE 7-16. (The ratio of the nominal wind speeds using the ASCE 7 prior to 2010 and the ASCE 7-16 varies somewhat depending on the site conditions but is about 1.6, which is the difference in the load factors.) If the nominal wind load W is based on of wind speeds for return periods of the order of 100 years, the load factor for W should be 1.6.

- For extreme load combination NB2-9, the load factor of 0.7 in N690 is appropriate for steel and SC structures. N690 invokes the principal action/companion action method of combining loads probabilistically. The principal loads in load combination NB2-9 are the accident and impact loads which are taken at their peak maximum values while Es is a companion load and the probability of simultaneous occurrences of the peak Es with the accident and impact loads is small. Even for those low probability cases when Es and accident loads would be concurrent, the time scales of Es and the accident loads are different; the peak response to SSE ground motion is reached in seconds while the peak response to an accident like a LOCA takes longer to reach, which leads to even smaller probabilities of coincident peak responses.

Other comments:

- Design for loads due to accidental explosions, or accidental vehicle impacts, or small aircraft impacts should use load combination (NB2-7) with those loads in lieu of Wt. The Wt load in the load combination (NB2-7) should be evaluated for tornado and for hurricane loads applicable to the site for which related regulatory guidance is in RG 1.76 and RG 1.221, respectively. RG 1.217 addresses effects from beyond-design-basis large aircraft impacts.

- When dead loads stabilize the structure the load factor should be 0.9 rather than 1.2 (comment 5d(4) in N690 has a strong technical basis and has been substantiated by reliability analysis.

- Except if otherwise justified, the hydrostatic fluid loads (F) should be zero when they act against destabilizing effects or uplift (supplements commends 5d(3) and 5d(4) in N690).

- Except if otherwise justified, the load factor for the weight and pressure of soil, water in soil, or weight and pressure of other materials, should be zero if the structural action due to these loads counteracts that from W, Wt, Eo or Es. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

- Differential settlements – The load factor for differential settlements should be the same as that for dead loads. However, differential settlement is a self-straining structural action while the dead load is a force-controlled load. The design should also account for this difference between differential settlement and force-controlled loads.

Proposed ASD Load Combinations for the DG endorsing N690-18

This section provides the development of ASD load combinations for possible guidance. Table 3 has the ASD load combinations in N690-18.

Table 3. Allowable Strength Design (ASD) Load Combinations in AISC N690-18

AISC N690-18 Section NB2 ASD*		**Load, Eqn. #**
U = (D + F) + (L + H + Ro + To)	+ C	NL, (NB2-10)
U = (D + F) + (H + Ro + To)	+ C + (Lr or S or R)	NL, (NB2-11)
U = (D + F) + (0.75L + 0.75H + To)	+ C + 0.75(Lr or S or R)	NL, (NB2-12)
U = (D + F) + (0.75L + 0.75H + Ro + To) + 0.6W	+ C + 0.75(Lr or S or R)	SEL, (NB2-13)
U = (D + F) + (0.75L + 0.75H + Ro + To) + Eo	+ C + 0.75(Lr or S or R)	SEL, (NB2-14)
U = D + F + L + H + To + Ro + Es	+ C	EEL, (NB2-15)
U = D + F + L + H + To + Ro + Wt		EEL, (NB2-16)
U = D + F + L + H + Ta + Ra + Pa	+ C	AL, (NB2-17)
U = D + F + L + H + Ta + Ra + Pa + Yr + Yj +Ym + 0.7Es		AL, (NB2-18)

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

The SEL, EEL and AL load combinations for the ASD method align well with those for the LRFD method in N690-18. However, the NL load combinations, NB2-10 through NB2-12 are not consistent with load combinations NB2-1 through NB2-3. Given the general AISC approach for the development of the LRFD and the subsequent development of the new ASD approach, the NL load combinations should include the same loads for the ASD and LRFD methods. In addition, the principal loads in the equivalent load combinations should be same. NB2-1 is essentially a dead load combination, NB2-2 is essentially a live load combination and NB2-3 is essentially a live load combination but with the roof/snow/rain loads as the principal loads. For this reason, NB2-10 should be modified to remove L and H, NB2-11 should be modified to add L and NB2-12 should be modified to add Ro. In this manner it is possible to compare the LRFD load combinations proposed for the DG with those for the ASD in N690-18 and propose modifications to the load factors for consistency between the load combinations for both methods.

In addition to modifying the NL load combinations for the ASD method to have the same loads as those for the LRFD method, a few load factors for all ASD load combinations in the N690-18 also had to be modified to be consistent with the LRFD load combinations proposed in the previous section. The resulting ASD load combinations proposed for this DG are in Table 4.

Table 4. Proposed ASD and LRFD Load Combinations

Proposed Guidance for DG	**ASD**[*,**]	**Load, Eqn. #**
U = (D + F) + (Ro + To)	+ C	NL, (NB2-10)
U = (D + F) + (**L** + H + Ro + To)	+ C + **0.75**(Lr or S or R)	NL, (NB2-11)
U = (D + F) + (0.75L + 0.75H + **0.75Ro** + To)	+ C + **1.0**(Lr or S or R)	NL, (NB2-12)
U = (D + F) + (**1.0**L + **1.0**H + Ro + To) + 0.6W	+ C + 0.75(Lr or S or R)	SEL, (NB2-13)
U = (D + F) + (**1.0**L + **1.0**H + Ro + To) + Eo	+ C + 0.75(Lr or S or R)	SEL, (NB2-14)
U = D + F + L + H + To + Ro + Es	+ C	EEL, (NB2-15)[*]
U = D + F + L + H + To + Ro + Wt		EEL, (NB2-16)[*]
U = D + F + L + H + Ta + Ra + Pa	+ C	AL, (NB2-17)[*]
U = D + F + L + H + Ta + Ra + Pa + Yr + Yj +Ym + 0.7Es[***]		AL, (NB2-18)[*]

[*] For Load Combinations NB2-15 through NB2-18 it is permitted to increase the allowable strength by 1.5, except for load combination NB2-17 for which the allowable stress increase factor is limited to 1.4 for consistency with the proposed guidance for NB2-8. This is a departure from the reduction by a factor of 1.6 in N690-18, except for members and fasteners in axial tension and shear. These changes increase the likelihood that the required capacity for the ASD approach is not less than that for the LRFD approach.
[**] The bold type font in this table highlights the differences between the AISC N690-18 ASD load combinations and the recommended ASD load combinations.
[***] If using 0.7Es in NB2-18, the SRSS approach is not permitted to combine dynamic loads. If using 1.0Es in NB2-18, then SRSS is permitted.

Proposed Guidance for DG	**LRFD**[*]	**Load, Eqn. #**
U = 1.4(D + F) + (**1.0**Ro + To)	+ 1.0C	NL, (NB2-1)
U = 1.2(D + F) + (1.6L + 1.6H + **1.6**Ro + *1.2*To)	+ 1.4C + 0.5(Lr or S or R)	NL, (NB2-2)
U = 1.2(D + F) + (0.8L + 0.8H + **0.8**Ro + *1.2*To)	+ 1.4C + *1.6(*Lr or S or R)	NL, (NB2-3)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.0W	+ 1.0C + 0.5(Lr or S or R)	SEL, (NB2-4)
U = 1.2(D + F) + (**1.6**L + 1.6H + **1.6**Ro + To) + 1.6Eo	+ 1.0C + 0.2(Lr or S or R)	SEL, (NB2-5)
U = D + F + **1.0**L + H + To + Ro + Es	+ C	EEL, (NB2-6)
U = D + F + **1.0**L + H + To + Ro + Wt		EEL, (NB2-7)
U = D + F + **1.0**L + H + Ta + Ra + **1.4**Pa	+ C	AL, (NB2-8)
U = D + F + **1.0**L + H + Ta + Ra + Pa + Yr + Yj + Ym + 0.7Es[**]		AL, (NB2-9)

[*] The bold type font in this table highlights the differences between the AISC N690-18 LRFD load combinations and the recommended LRFD load combinations.
[**] If using 0.7Es in NB2-18, the SRSS approach is not permitted to combine dynamic loads. If using 1.0Es in NB2-18, then SRSS is permitted.

- The load 1.0L in NB2-11 is inserted to be consistent with the 1.6L in NB2-2. The load factor for the roof loads in NB2-11 is reduced to 0.75 because these are not the principal live loads in this load combination. The roof loads are the principal live loads in load combination NB2-12 for which the load factor was increased from 0.75 to 1.0
- In NB2-13 and NB2-14, the load factors for L and H in the SEL loads are increased to 1.0 for consistency with the load factors of 1.6 for L and H in the corresponding recommended LRFD load combinations (NB2-4 and NB2-5).

The section at the end of this Appendix titled 'Tables Illustrating ASD and LRFD Load Combinations Comparisons' has tables that illustrate the relative values of required capacities obtained with load combinations for the LRFD and ASD (allowable strength design) approaches in N690-18 and those recommended for Draft Guide 1304. The tables in that section assume nominal values for the various loads in terms of multiples of the nominal dead load, D.

As an illustration, the load combinations for the allowable stress design in the 2002 Supplement 2 to N690-1994 are as shown in Table 5. These load combinations for the allowable stress approach are not directly comparable to those for the allowable strength approach in N690-18. However, the allowable stress design load combinations resemble those for the allowable strength design method. A noticeable difference is that F and H loads were not explicitly considered in the allowable stress method. The wind load W did not use the 0.6 factor because the associated return period for the nominal load was 100 years as opposed to 3000 years in the N690-18 approach. The allowable stress load combinations combine Eo with accident loads. Because under the operating basis earthquake the plant is expected to continue to operate it does not seem justified to combine Eo with a design basis event that would lead to a plant shutdown (load combination AL, SEL, 11 in Table 5).

Table 5. Load Combinations for the Allowable Stress Approach in Supplement 2 to N690-1994

AISC N690-1994-s2004 ASD [f]	Load, Eqn. #	Stress Limit Coefficient[b, h]
U = D + L	NL, 1	1.0[c]
U = D + L + Ro + To	NL, 2	1.0[c]
U = D + L + W	SEL[i], 3	1.0[c]
U = D + L + Eo	SEL[i], 4	1.0[c]
U = D + L + Ro + To + W	SEL[i], 5	1.0[c]
U = D + L + Ro + To + Eo	SEL[i], 6	1.0[c]
U = D + L + Ro + To + Wt	EEL, 7	1.6[g,k]
U = D + L + Ro + To + Es	EEL, 8	1.6[g,k]
U = D + L + Ra + Ta + Pa	AL[d], 9	1.6[g,k]
U = D + L + Ta + Pa	AL[d,j], 9a	1.6[g,k]
U = D + L + Ra + Ta + Pa + Yr + Yj + Ym + Eo	AL[d,e], SEL, 10	1.6[g,k]
U = D + L + Ra + Ta + Pa + Yr + Yj + Ym + Es	AL[d,e], SEL, 11	1.7[g,k]

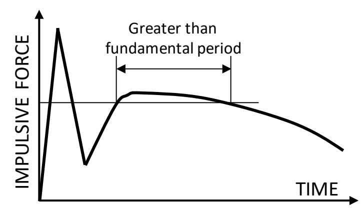
a. Coefficients are applicable to primary stress limits in Sections Q.1.5.1, Q.1.5.2, Q.1.5.3, Q.1.5.4, Q.1.5.5, Q.1.5.6, Q.1.10 and Q.1.11 of N690-1994 and N690-1994, Supplement 2 (2002).
b. In no instance shall the allowable stress exceed 0.7Fu in axial tension nor 0.7Fu times the Z/S for tension plus bending.
c. For primary plus secondary stress, the allowable limits are increased by a factor of 1.5
d. The maximum value of Pa, Ta, Ra, Yj, Yr, and Ym, including an appropriate dynamic load factor, shall be used in load combinations 9 through 11, unless appropriate time history analysis is performed to justify otherwise.
e. In combining loads for from a loss of coolant accident (LOCA) and a seismic event the SRSS (square root of the sum of squares) may be used, provided that the responses are calculated on a linear basis.
f. All load combinations shall be checked for a no-live-load condition.
g. In load combinations 7 through 11, the stress limit coefficient in shear shall not exceed 1.4 in members and bolts.
h. Secondary stresses which are used to limit primary stresses shall be treated as primary stresses.
i. Consideration shall also be given to snow and other loads as defined in ASCE 7.
j. This load combination is to be used when the global (non-transient) sustained effects of Ta are considered.
k. The stress limit coefficient where axial compression exceeds 20% of normal allowable, shall be 1.5 for load combinations 7, 8, 9, 9a and 10 and 1.6 for load combination 11.

DESIGN FOR IMPACTIVE AND IMPULSIVE LOADS

Section N9.1.6 of ANSI/AISC N690-18, 'Design for Impactive and Impulsive Loads,' and its subsections identify the criteria and provisions for design of SC walls for impactive and impulsive loads. This section presents the required design criteria and ductility ratios for flexure-controlled SC walls, shear-controlled SC walls and SC walls under axial compressive loads. As explained in the Commentary to Section N9.1.6, the section is based on ACI 349-13, Appendix F, 'Special Provisions for Impulsive and Impactive Effects.' While the criteria and provisions in the section are specific to SC walls, some of the criteria in N690-18 are not consistent with those in ACI 349-13 Appendix F and NRC Regulatory Guide 1.142 Rev. 3 for reinforced concrete members. Table 6 identifies differences between the criteria and provisions in N690-18, ACI 349-13 and RG 1.142 Rev. 3 as well as recommendations where regulatory positions should be developed to provide consistency in the guidance for the design for impactive and impulsive loads for SC walls and RC walls.

Table 6. Comparison of Provisions for Design for Impulsive and Impactive Loads

Item and whether a regulatory position is proposed	**N690**	**ACI 349-13 and RG 1.142 Rev. 3**
Dynamic Increase Factors (DIF)	**N9.1.6a** Table A-N9.1.1, with the same values as those in ACI 349-13 Only grades 40 and 60 for reinforcing steel	<u>ACI 349</u> **F.2** (concrete strength between 4000 and 6000 psi) • F.2.1 – Steel Reinforcement (1.2 for Grade 40, 1.15 for Grade 50 and 1.1 for Grade 60) – Similar to N690 Table A-N9.1.1 but includes Grades above 60 • F.2.1 – Concrete – axial and flexural compression 1.25; shear 1.10 • F.2.1 – Limits DIF to 1.0 when the dynamic load factor (DLF) used is less than 1.2. <u>RG 1.142</u> Limits the DIF to 1.0 per Table 5.4 in ASCE (1980), 'Report of the Committee on Impactive and Impulsive loads, Volume 5,' for: • Diagonal tension and direct shear for reinforcing steel (stirrups) • Diagonal tension and direct shear (punchout) and bond for reinforced and prestressed concrete (Should be verified against more recent ASCE publications on this topic)
Yield displacement and force for flexure	**N9.1.6a** Specific to SC walls	<u>ACI 349</u> Yield displacement defined in **F.3.1.**

Item and whether a regulatory position is proposed	N690	ACI 349-13 and RG 1.142 Rev. 3
		Loss of intended function also important also shall be verified in addition to the deformation limits in F.3.3 and F.3.4
Strength requirements for long duration impulsive loads (Impactive and impulsive loads shall be considered concurrent with other loads in determining the structural resistance **F.8**) **** Regulatory position is proposed**	Criteria not provided	<u>ACI 349</u> **F.3.2** - Strength available for impulsive loads shall be at least 20 percent greater than the magnitude of any portion of the impulsive loading, which is approximately constant for a time equal to or greater than the first fundamental period of the structural member. 
Criterion to define flexure-controlled members	**N9.1.6b** Requires that the available strength for flexure is 25% greater than that for shear	**F.3.6** – ACI requires that the available strength for flexure 20% greater than that for shear.
Ductility flexure-controlled **** Regulatory position is proposed**	**N9.1.6b** • 10 permissible (symmetric sections) • No rotational criteria (the commentary says it is not needed but the reference cited, Varma et al (2011c) does not provide a clear justification. Test data in Reference 11, support a rotational capacity of 0.07 radians (4 degrees) when flexure controls the design. • No consideration of axial forces or compartment pressurization • The ACI 349 rotational criteria may not be directly applicable • The commentary to N9.1.6c, says that the ductility should be	<u>ACI 349</u> • **F.3.3** limits to $0.05/(\rho - \rho')$ not to exceed 10 or shall be determined from the rotational capacity as defined in **F.3.4**, which limits rotations to 0.0065(d/c) but not exceed 0.07 radians (4 degrees) • **F.3.8** limits the allowable ductility depending on the magnitude of the compressive axial force with limits ranging from 1.3 to 10. <u>RG 1.142</u> Similar provisions but with the lower ductility limit being 1.0 instead of 1.3

Item and whether a regulatory position is proposed	N690	ACI 349-13 and RG 1.142 Rev. 3
	limited as defined in N9.1.6c or the plate principal strain limited to 0.05 as described in Johnson et al. (2014) (Ref. 8).	
Ductility for compartment pressurization or that could affect the integrity of the structure as a whole **** Regulatory position is proposed for consistency with ACI 349-13**	**N9.1.6b** No provisions	ACI 349 **F3.5** limits the ductility to 3.0 for loads such as blast or compartment pressurization which can affect the structure as a whole. RG 1.142 Provides additional criteria
Ductility shear controlled **** Regulatory position is proposed**	**N9.1.6b** • Up to 1.6 (with yielding shear reinforcement spaced at less than or equal to half the wall thickness (t_s/2) • Other configurations with yielding or non-yielding up to 1.3 • There is reinforcement always	ACI 349 **F.3.7** • Shear resisted by concrete alone, 1.3 • Shear resisted by concrete and stirrups or bent bars, 1.6 • Shear completely resisted by stirrups, 3.0 RG 1.142 Uses 1.0 for the first case and 1.3 for the other two cases.
Ductility axial compression impulsive or impactive loads **** Regulatory position is proposed**	**N9.1.6b** Up to 1.3	ACI 349 **F.3.9** limits the ductility to 1.3 RG 1.142 Limits the ductility to 1.0
Requirements to ensure ductility	Addressed in the overall SC wall requirements in N.9.1 in a manner specific to SC walls.	ACI 349 **F.4** • F.4.1 – concrete strength ≥ 3,000 psi • F.4.1 – Reinforcement yield strength ≤ 80,000 psi. Grade and area of flexural reinforcement shall be only as specified. RG 1.1.42 Rev. 3 does not endorse, in general, the use of high strength reinforcement (Grade 75 and 80) that is permitted in ACI 349-13. • F.4.2 and F.4.3 place conditions on details of reinforcement • F.4.4 – Vertical reinforcement ratio for

Item and whether a regulatory position is proposed	N690	ACI 349-13 and RG 1.142 Rev. 3
		columns to be between 1.0% and 6.0%. • F4.5 – Confinement reinforcement for columns
Shear Strength **** Regulatory position is proposed for clarification** In the case of the reaction shear (beam action condition) at the supports, the effective width of the critical section for the shear beam capacity at the supports is to be determined according to the zone of influence induced by the local loads instead of the entire width of the support. The zone of influence induced by the concentrated loads may be determined, for example, by an analysis.	Not provided	<u>ACI 349</u> **F.5** • Account for punching shear adjacent to the load and reaction shear at the supports for impulsive and impactive loads except that for impactive loads criteria for perforation shall be used in place of punching shear requirements • The shear strength is determined in accordance with the principles for static loads (11.1, 11.2 and 11.11 for punching shear) increase by the applicable DIF. <u>RG 1.142</u> RP 4.8 and RP 6.4.7 address the effective width for beam action condition for the shear strength of walls for concentrated loads or reactions perpendicular to the plane of the walls. Instead of the entire width of the slab (as specified in Section 11.11.1.1 of ACI 38-08, which is incorporated by reference in Section 11.9 of ACI 349-3), the width of the critical section for the beam action condition is the effective zone of influence induced by the concentrated loads. The effective zone of influence may be determined, for example, by analysis.
Response determination for impulsive loads **** Regulatory position is proposed for clarification for applicability of response methods to impactive loads (ACI F.7.3 in the row below)**	**N9.1.6c** N690 provisions are the same as in ACI 349 • The applicable ductility limits should be those defined in the new DG	**F.6** • **F.6.1** – impulsive loads shall be considered in combination with other loads refers to **F.8** • **F.6.2**(a) – DLF with ductility criteria • **F.6.2**(b) – Impulse, momentum and energy balance with strain energy limited by the ductility limits • **F.6.2**(c) – Time-history dynamic analysis with permissible ductility limits
Impactive loads	**N9.1.6c** • Design for impact loads shall satisfy the criteria for both local and overall	**F.7** • **F.7.1** – Criteria for both local and overall structural response • **F.7.2** – Local impact effects included

Item and whether a regulatory position is proposed	N690	ACI 349-13 and RG 1.142 Rev. 3
	loads, and local effects shall include perforation of the SC wall • Faceplate thickness to prevent perforation at least 25% greater than calculated using rational methods • Refers to the commentary for rational methods Criteria for perforation is used in place of punching shear requirements as in ACI F.5.	penetration, perforation, scabbing and punching shear • F.7.2.1 – Penetration depth, perforation depth and required thickness – Use applicable formulas or test data; for perforation prevention the concrete thickness shall be 20% than that required to prevent perforation • F.2.7.2 – Scabbing prevention (section 20% thicker than that necessary to prevent scabbing or provisions for scabbing shields0 • F.7.2.3 – Punching shear design by F.5 not need when perforation approach as per F.7.2.1 is used • F.7.2.4 – Minimum slab or wall reinforcement • **F.7.3** – Structural response determined by the methods in **F.6.2**
Impactive and Impulsive Loads **** Regulatory position is proposed**	Only refers to this criterion in the commentary.	**F.8** Impactive and impulsive loads shall be considered concurrent with other loads (for example dead and live load) in determining the required resistance of structural members.

The following are tentative provisions for impactive and impulsive loads for SC walls recommended for the DG.

ANSI/AISC N690-18 Appendix N9 – Steel-Plate Composite (SC) Walls

ANSI/AISC N690-8, Appendix N9, Section N9.1.6 – Design for Impactive and Impulsive loads

Section N9.1.6 and its subsections are based on ACI 349-13, Appendix F, Special Provision for Impulsive and Impactive Effects adapted to SC walls. The staff considers the provisions in Section N9.1.6 and its subsections acceptable with the additions and exceptions proposed below.

1. Impactive and impulsive loads are assumed to be concurrent with other loads (e.g., dead and live loads) in determining the required strength of structural elements.

2. For impulsive loads, the strength available for impulsive loads is at least 20 percent greater than the magnitude of any portion of the impulsive loading, which is approximately constant for a time equal to or greater than the first fundamental period of the structural member.

3. In addition to the deformation limits under 4. to 7. below, the maximum deformation shall not result in the loss of intended function of the structural wall nor impair the safety-related function of other systems and components.

4. For flexure-controlled SC walls as defined in Section N9.6b of ANSI/AISC N690-18, the permissible displacement ductility ratio demand should satisfy all of the following:

   - ductility ratio less than or equal to 10,
   - principal strain of the faceplates less than or equal to 0.05 (Johnson et al., 2014), and
   - rotational capacity of any yield hinge less than or equal to 0.07 radians (4 degrees) [Bruhl et al., 2017).

5. For SC walls resisting axial compression, the permissible displacement ductility ratio should be as shown in 5.1 to 5.3.

   5.1 When compression controls the design as defined by the balanced point in a load-moment interaction diagram, the permissible ductility ratio shall be 1.0.
   5.2 When the compression load does not exceed 0.1(f'_c A_g), where A_g is the sum of the area of concrete infill and the net area of the faceplates, or one-third of that which would produce balanced strain conditions, whichever is smaller, the permissible ductility ratio should be as given in 1.1.4.
   5.3 The ductility ratio varies linearly between 1.0 and that given in 4 for conditions between those described in 5.1 and 5.2.

6. The permissible displacement ductility ratio in flexure should not exceed 3.0 for loads such as blast and compartment pressurization, which could affect the integrity of the structure as a whole.

7. For shear-controlled SC walls with yielding reinforcement spaced at section thickness divided by two or smaller, the ductility ratio is limited to 1.3. For shear-controlled SC walls with other configurations of yielding or nonyielding reinforcement, the ductility ratio is limited to 1.0.

8. The shear strength under local loads considers reaction shear at the supports and punching shear adjacent to the load.

   8.1 Local loads may be impulsive or impactive, except that for impactive loads, satisfaction of criteria for perforation shall be used in place of punching shear requirements.
   8.2 The shear strength is determined in accordance with the provisions in Section N9.3, 'Design of SC Walls,' of N690-18 using the appropriate dynamic increase factors (DIFs) in Table A-N9.1.1 for the required concrete and steel material properties.
   8.3 In the case of the reaction shear (beam action condition) at the supports, the effective width of the critical section for the shear beam capacity at the supports is to be determined according to the zone of influence induced by the local loads instead of the entire width of the support. The zone of influence induced by the concentrated loads may be determined, for example, by an analysis (see Ref. 9).

9. Design of SC walls or SC structural wall systems for impactive loads satisfies the criteria for local effects and overall structural response. Their structural response is determined by the methods for impulsive loads in Section N.9.1.6c. Local effects include penetration, perforation and punching shear. The penetration depth and required concrete and faceplate thickness required to prevent penetration are from applicable rational methods or pertinent test data.

10. Evaluation of loads from malevolent, beyond-design-basis, aircraft impacts are in the scope of RG 1.127 and outside the scope of this RG.

TABLES ILLUSTRATING ASD AND LRFD REQUIRED CAPACITIES

The tables in this section illustrate the ratio of required capacities obtained when using the LRFD and ASD load combinations in N690-18 and those recommended for the Draft Guide 1304 with assumptions about the relative values of the various nominal loads as multiples or fractions of the dead load, D.

Table 7. LRFD and ASD normal load conditions

Load Combination NB2-1 (Dead Loads) and NB2-10																								
Factor			D	gD	F	gF	L	gL	H	gH	Ro	gRo	T	gTo	C	gC	Roof	gRoof			Ru	Rn	ASD/LRFD	
0.9	Rn=Ru/phi	LRFD	1	1.4	0.5	1.4	0	0	0	0	0.5	1.4	1	1	1	1	0	0			4.8	5.3333		NB2-1 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	1	1	1	0.5	1	1	1	1	1	0	0			6	10.02	1.879	NB2-10 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	0	**0**	0	**0**	0.5	1	1	1	1	1	0	0			4	6.68	1.253	NB2-10 - DG
0.9	Rn=Ru/phi	LRFD	1	1.4	0.5	1.4	0	0	0	0	0.5	**1**	1	1	1	1	0	0			4.6	5.1111		NB2-1 - DG
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	1	1	1	0.5	1	1	1	1	1	0	0			6	10.02	1.960	NB2-10 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	0	**0**	0	**0**	0.5	1	1	1	1	1	0	0			4	6.68	1.307	NB2-10 - DG

Load Combination NB2-2 (Live Loads) and NB2-11																								
Factor			D	gD	F	gF	L	gL	H	gH	Ro	gRo	To	gTo	C	gC	Roof	gRoof			Ru	Rn	ASD/LRFD	
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	1.6	1	1.6	0.5	1.2	1	1.2	1	1.4	0.5	0.5			11.65	12.944		NB2-2 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	0	0	1	1	0.5	1	1	1	1	1	1	1			6	10.02	**0.774**	NB2-11 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	**1**	1	1	0.5	1	1	1	1	1	1	**0.75**			8.75	14.613	1.129	NB2-11 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	1.6	1	1.6	0.5	**1.6**	1	1.2	1	1.4	0.5	0.5			11.85	13.167		NB2-2 - DG
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	0	0	1	1	0.5	1	1	1	1	1	1	1			6	10.02	**0.761**	NB2-11 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	**1**	1	1	0.5	1	1	1	1	1	1	**0.75**			8.75	14.613	1.110	NB2-11 - DG

Load Combination NB2-3 (Lr or S or R) and NB2-12																								
Factor			D	gD	F	gF	L	gL	H	gH	Ro	gRo	To	gTo	C	gC	Roof	gRoof			Ru	Rn	ASD/LRFD	
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	0.8	1	0.8	0.5	1.2	1	1.2	1	1.4	1	1.6			9.8	10.889		NB2-3 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0	0	1	1	1	1	1	0.75			7.25	12.108	1.112	NB2-12 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0.5	**0.75**	1	1	1	1	1	**1**			7.875	13.151	1.208	NB2-12 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	0.8	1	0.8	0.5	**0.8**	1	1.2	1	1.4	1.5	1.6			10.4	11.556		NB2-3 - DG
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0	0	1	1	1	1	1.5	0.75			7.625	12.734	1.102	NB2-12 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0.5	**0.75**	1	1	1	1	1.5	**1**			8.375	13.986	1.210	NB2-12 - DG

Table 8. LRFD and ASD Severe Environmental Load Conditions with Eo as the Dominant Load

Load Combination NB2-5 (Eo) and NB2-14 (Similar considerations apply to load combinations NB2-4 and NB2-13)																									
Factor			D	gD	F	gF	L	gL	H	gH	Ro	gRo	To	gTo	Eo	gEo	C	gC	Lr	gLr		Ru	Rn	ASD/ LRFD	
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	0.8	1	1.6	0.5	1.2	1	1	1	1.6	0.5	1	0.5	0.2		9.6	10.667		NB2-5 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0.5	1	1	1	1	1	0.5	1	0.5	0.75		7.875	13.151	1.233	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	**1**	1	**1**	0.5	1	1	1	1	1	0.5	1	0.5	0.75		8.875	14.821	1.389	NB2-14 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	**1.6**	1	1.6	0.5	**1.6**	1	1	1	1.6	0.5	1	0.5	0.2		11.6	12.889		NB2-5 - DG
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0.5	1	1	1	1	1	0.5	1	0.5	0.75		7	11.690	**0.907**	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	**1**	1	**1**	0.5	1	1	1	1	1	0.5	1	0.5	0.75		8	13.360	1.037	NB2-14 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	1	0.8	1	1.6	0.5	1.2	1	1	1	1.6	0.5	1	0.5	0.75		7.4	8.222		NB2-5 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	0.75	1	0.75	0.5	1	1	1	1	1	0.5	1	0.5	0.75		5.5	9.185	1.117	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	**1**	1	**1**	0.5	1	1	1	1	1	0.5	1	0.5	0.75		6	10.020	1.219	NB2-14 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	1	**1.6**	1	1.6	0.5	**1.6**	1	1	1	1.6	0.5	1	0.5	0.75		8.4	9.333		NB2-5 - DG
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	0.75	1	0.75	0.5	1	1	1	1	1	0.5	1	0.5	0.75		5.5	9.185	**0.984**	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	**1**	1	**1**	0.5	1	1	1	1	1	0.5	1	0.5	0.75		6	10.020	1.074	NB2-14 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	1	**1.6**	1	1.6	0.5	**1.6**	1	1	1.5	1.6	0.5	1	0.5	0.75		9.2	10.222		NB2-5 - DG
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	0.75	1	0.75	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		6	10.020	**0.980**	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	**1**	1	**1**	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		6.5	10.855	1.062	NB2-14 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	1	0.8	1	1.6	0.5	1.2	1	1	1.5	1.6	0.5	1	0.5	0.75		8.2	9.111		NB2-5 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	0.75	1	0.75	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		6	10.020	1.100	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	1	**1**	1	**1**	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		6.5	10.855	1.191	NB2-14 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	**1.6**	1	1.6	0.5	**1.6**	1	1	1.5	1.6	0.5	1	0.5	0.75		12.4	13.778		NB2-5 - DG
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		7.5	12.525	**0.909**	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	**1**	1	**1**	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		8.5	14.195	1.030	NB2-14 - DG
0.9	Rn=Ru/phi	LRFD	1	1.2	0.5	1.2	3	0.8	1	1.6	0.5	0.8	1	1	1.5	1.6	0.5	1	0.5	0.75		9.6	10.667		NB2-5 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	0.75	1	0.75	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		7.5	12.525	1.174	NB2-14 - N690
1.67	Rn=Ru(Om)	ASD	1	1	0.5	1	3	**1**	1	**1**	0.5	1	1	1	1.5	1	0.5	1	0.5	0.75		8.5	14.195	1.331	NB2-14 - DG

The first column in Table 9 to Table 11 uses a resistance factor of 0.9 for the line with LRFD load combinations. For the lines for the ASD load combinations, the first column has the ratio of the ASD factor of safety of 1.67 to a factor that multiplies the allowable stress (i.e., reduces the factor of safety). For the ASD load combinations, the quantity in the first column of Table 9 to Table 11, multiplies the factored load effect Ru to obtain the required nominal capacity, Rn. The allowable stress multiplier is 1.6 for the N690-18 load combinations (1.67/1.6=1.0438 in column 1) and 1.5 for the recommended load combinations (1.67/1.5=1.1133 in column 1). For the ASD load combinations with accident pressure (NB2-8 and NB2-17) in Table 10, an additional line is added for an allowable stress multiplier of 1.4 (1.67/1.4=1.1929 in column 1).

Table 9. LRFD and ASD Extreme Environmental Load Conditions with Es as the Dominant Load

Load Combination NB2-6 (Ess) and NB12-15 (Similar considerations apply to NB2-7 and NB2-16)																										
Factor			D	gD	F	gF	L	gL	H	gH	Ro	gRo	To	gTo	Es	gEs	C	gC				Ru	Rn	ASD/ LRFD		
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	0.8	1	1	0.5	1	1	1	1	1	0.5	1				7.9	8.778		NB2-6 - N690	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	0.5	1				8.5	8.872	1.011	NB2-15 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	0.5	1				8.5	9.463	1.078	NB2-15 - DG	
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	**1**	1	1	0.5	1	1	1	1	1	0.5	1				8.5	9.444		NB2-6 - DG	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	0.5	1				8.5	8.872	0.939	NB2-15 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	0.5	1				8.5	9.463	1.002	NB2-15 - DG	
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	0.8	1	1	0.5	1	1	1	2	1	0.5	1				8.9	9.889		NB2-6 - N690	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	0.5	1				9.5	9.916	1.003	NB2-15 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	0.5	1				9.5	10.577	1.070	NB2-15 - DG	
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	**1**	1	1	0.5	1	1	1	2	1	0.5	1				9.5	10.556		NB2-6 - DG	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	0.5	1				9.5	9.916	0.939	NB2-15 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	0.5	1				9.5	10.577	1.002	NB2-15 - DG	

Table 10. LRFD and ASD Abnormal Load Combination with Accident Pressure, Pa

Load Combination NB2-8 (Pa) and NB2-17																										
Factor			D	gD	F	gF	L	gL	H	gH	Ra	gRa	Ta	gTa	Pa	gPa	C	gC				Ru	Rn	ASD/ LRFD		
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	0.8	0.5	1	0.5	1	1	1	1	1.2	0.5	1				7.6	8.444		NB2-8 - N690	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	0.5	1	0.5	1	1	1	1	1	0.5	1				8	8.350	**0.989**	NB2-17 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	0.5	1	0.5	1	1	1	1	1	0.5	1				8	8.907	1.055	NB2-17 - DG	
1.1929	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	0.5	1	0.5	1	1	1	1	1	0.5	1				8	9.543	1.130	NB2-17 - DG	
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	**1**	0.5	1	0.5	1	1	1	1	**1.4**	0.5	1				8.4	9.333		NB2-8 - DG	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	0.5	1	0.5	1	1	1	1	1	0.5	1				8	8.350	**0.895**	NB2-17 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	0.5	1	0.5	1	1	1	1	1	0.5	1				8	8.907	**0.954**	NB2-17 - DG	
1.1929	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	0.5	1	0.5	1	1	1	1	1	0.5	1				8	9.543	1.022	NB2-17 - DG	
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	2	0.8	0.5	1	0.5	1	1	1	2	**1.4**	0.5	1				8.4	9.333		NB2-8 - DG	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	2	1	0.5	1	0.5	1	1	1	2	1	0.5	1				8	8.350	**0.895**	NB2-17 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	2	1	0.5	1	0.5	1	1	1	2	1	0.5	1				8	8.907	**0.954**	NB2-17 - DG	
1.1929	Rn=Ru(Om)	ASD	1	1	0.5	1	2	1	0.5	1	0.5	1	1	1	2	1	0.5	1				8	9.543	1.022	NB2-17 - DG	
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	2	**1**	0.5	1	0.5	1	1	1	2	**1.4**	0.5	1				8.8	9.778		NB2-8 - DG	
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	2	1	0.5	1	0.5	1	1	1	2	1	0.5	1				8	8.350	**0.854**	NB2-17 - N690	
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	2	1	0.5	1	0.5	1	1	1	2	1	0.5	1				8	8.907	**0.911**	NB2-17 - DG	
1.1929	Rn=Ru(Om)	ASD	1	1	0.5	1	2	1	0.5	1	0.5	1	1	1	2	1	0.5	1				8	9.543	0.976	NB2-17 - DG	

Table 11. LRFD and ASD Abnormal Load Conditions with Accident Loads Y as the Dominant Loads

Load Combination NB2-9 (AL) and NB12-18																												
Factor			D	gD	F	gF	L	gL	H	gH	Ra	gRa	Ta	gTa	Pa	gPa	Yr	gYr	Yi	gYi	Ym	gYm	Es	gEs	Ru	Rn	ASD/ LRFD	
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	0.8	1	1	0.5	1	1	1	1	1	1	1	1	1	1	1	1	0.7	11.1	12.333		NB2-9 - N690
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	1	1	1	1	1	1	1	0.7	11.7	12.212	**0.990**	NB2-18 - N690
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	1	1	1	1	1	1	1	0.7	11.7	13.026	1.056	NB2-18 - DG
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	**1**	1	1	0.5	1	1	1	1	1	1	1	1	1	1	1	1	0.7	11.7	13.000		NB2-9 - DG
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	1	1	1	1	1	1	1	0.7	11.7	12.212	**0.939**	NB2-18 - N690
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	1	1	1	1	1	1	1	1	1	0.7	11.7	13.026	1.002	NB2-18 - DG
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	0.8	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	1	1	12.4	13.778		NB2-9 - N690, gEs=1.0
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	1	1	13	13.569	**0.985**	NB2-18 - N690, gEs=1.0
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	1	1	13	14.473	1.050	NB2-18 - DG, gEs=1.0
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	**1**	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	1	1	13	14.444		NB2-9 - DG, gEs=1.0
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	1	1	13	13.569	**0.939**	NB2-18 - N690, gEs=1.0
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	1	1	13	14.473	1.002	NB2-18 - DG, gEs=1.0
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	0.8	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	12.8	14.222		NB2-9 - N690
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	13.4	13.986	**0.983**	NB2-18 - N690
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	13.4	14.919	1.049	NB2-18 - DG
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	3	**1**	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	13.4	14.889		NB2-9 - DG
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	13.4	13.986	**0.939**	NB2-18 - N690
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	3	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	13.4	14.919	1.002	NB2-18 - DG
0.9	Rn=Ru/phi	LRFD	1	1	0.5	1	1	**1**	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	11.4	12.667		NB2-9 - DG
1.0438	Rn=Ru(Om)	ASD	1	1	0.5	1	1	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	11.4	11.899	**0.939**	NB2-18 - N690
1.1133	Rn=Ru(Om)	ASD	1	1	0.5	1	1	1	1	1	0.5	1	1	1	2	1	1	1	1	1	1	1	2	0.7	11.4	12.692	1.002	NB2-18 - DG

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