Enclosure 2 to E-56212

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1.2.2 Principal Design Criteria

The WCS CISF principal design criteria are based on the site characteristics, the design criteria associated with the cask systems listed in Table 1-1 that have been previously approved by the NRC, and specific criteria required for the WCS CISF design.

The cask systems listed in Table 1-1 meet the WCS CISF design criteria. Table 1-2 provides a summary of the WCS CISF principal design criteria.

1.2.3 Facility Descriptions

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The major facilities at the WCS CISF are the Cask Handling Building and the storage area. The Cask Handling Building is approximately *175* feet long by *193* feet wide by *72* feet high. The building is a two-bay steel structure designed to support two commercial overhead cranes used to move transportation casks from the rail car to the transport vehicle. One bay of the building will house the Canister Transfer System described in Section 1.3.1.2 and the other bay will be available for direct transfer of transportation casks from the rail car to the transport vehicle. A 2,400 square foot area of the building is set aside for cask storage. The building plan view is shown in Figure 1-7. Figure 1-8 is a section through the building showing the overhead crane location. Air monitors and dosimeters are located in the building for monitoring purposes. The building is not designed or intended to provide confinement or shielding for SNF or GTCC materials. The building is classified as ITS - Category *B*. The purpose of the Cask Handling Building is to receive and prepare for storage shipments of dual-purpose canister systems. It will also receive GTCC waste canisters for storage at the site. It is also designed to process canisters stored at the site for off-site shipment. The Cask Handling Building is designed to handle canisterized material and does not have the capability to handle bare fuel.

As Low As Reasonably Achievable (ALARA) principles are incorporated, to the maximum extent practical, throughout the facility design to reduce radiation exposure to facility personnel. Cranes/lifting devices for transferring the NUHOMS® transportation/transfer casks from the transportation skid to the transfer trailer/skid are designed to minimize the need for facility personnel to be near the loaded cask. This equipment is NITS as the lift heights of the loaded casks are maintained below 80 inches at all times after removal of the impact limiters. The analysis of bounding drop scenarios shows that a NUHOMS® transportation/transfer cask will maintain structural integrity of the DSC confinement boundary and maintain basket geometry from an 80 inch (from the bottom of the cask to the “ground”) drop. The ITS canister transfer system for the vertical transfer of canisters is remotely operated and the transfer equipment used to make the transfer to the storage overpacks is substantially identical to that used to transfer the canister into dry storage at the reactor facilities where the material was initially stored.

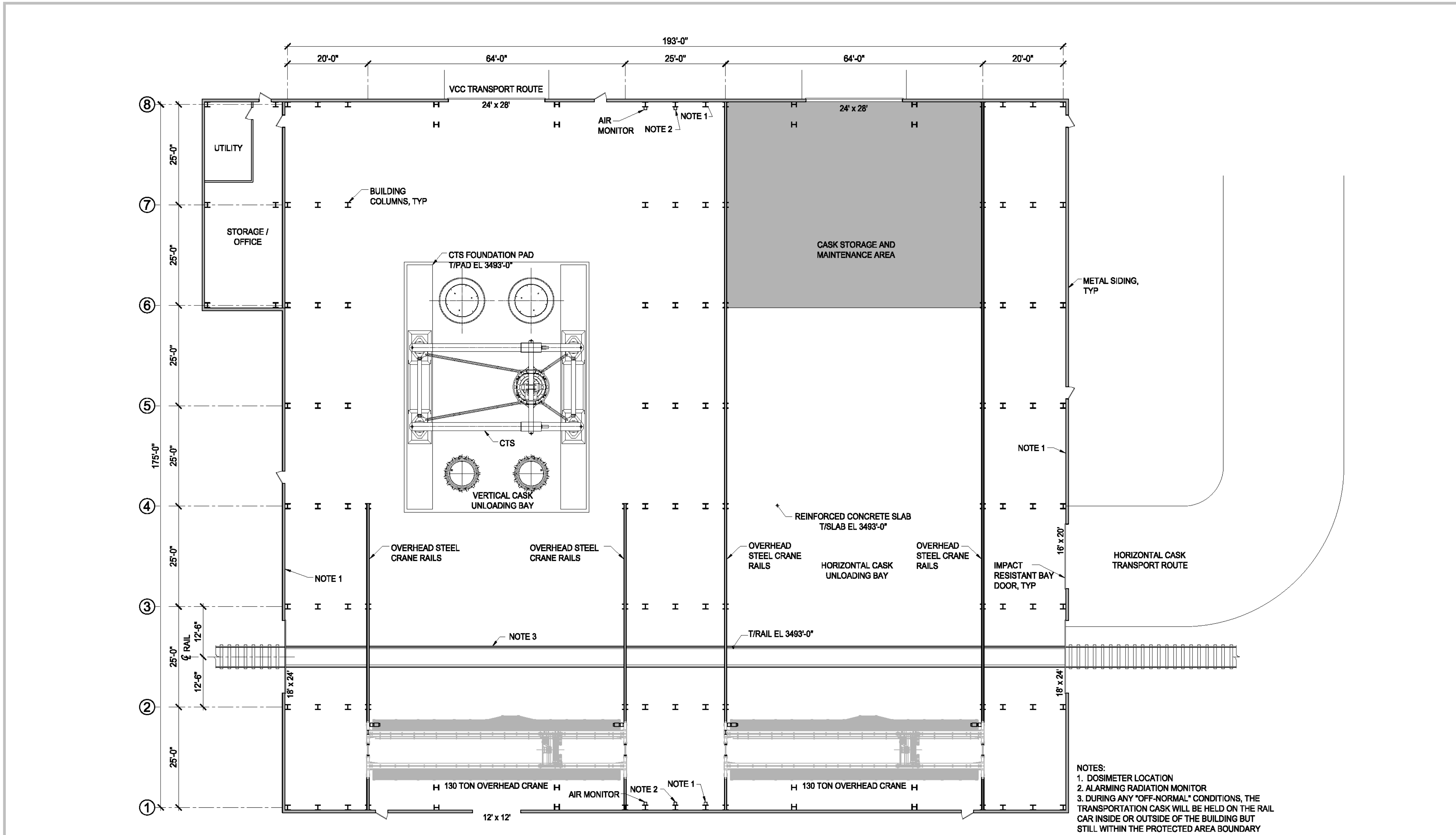

Figure 1-7
Cask Handling Building Plan

All Indicated Changes are in response to RAI NP-7-12

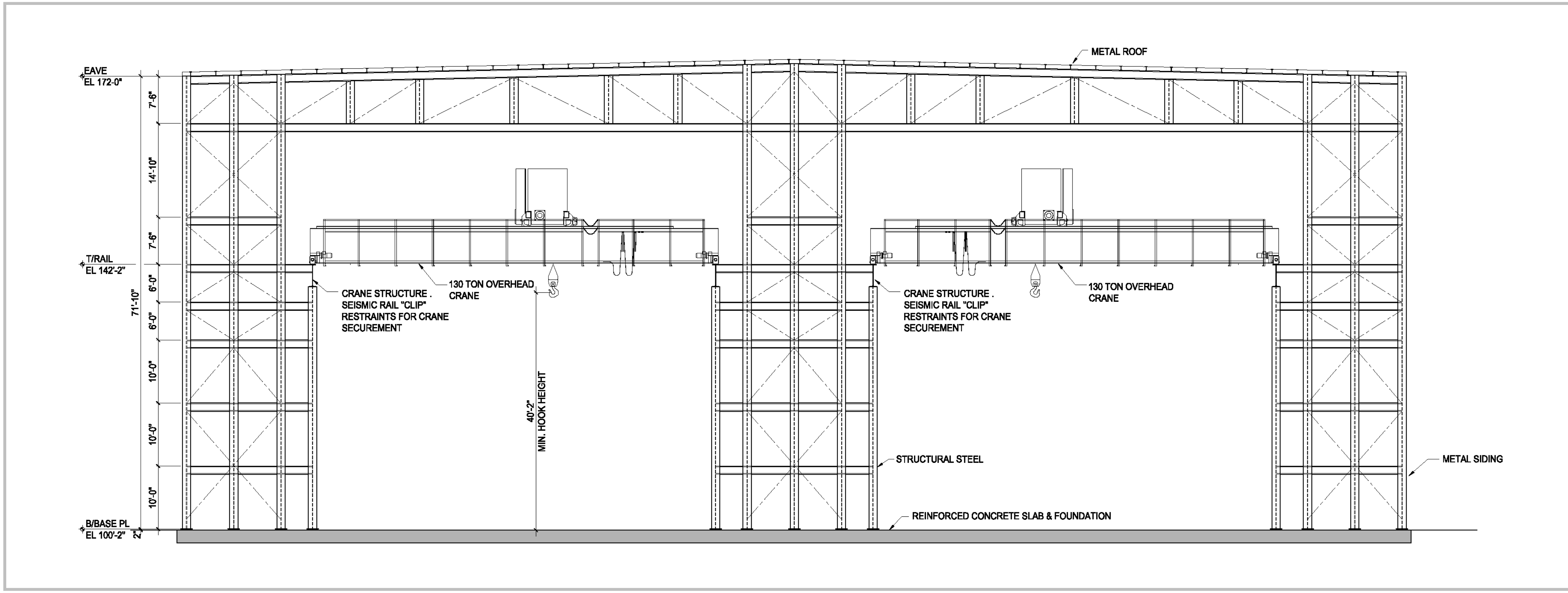

Figure 1-8
Cask Handling Building Section View

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3.2.2.2 Phenomena Considered in Design Load Calculations

SSCs are not in a floodplain and are above the PMF elevation. Therefore, they are not required to consider flood design loads.

3.2.2.3 Flood Force Application

SSCs are not in a floodplain and are above the PMF elevation. Therefore, they are not required to consider flood design loads.

3.2.2.4 Flood Protection

SSCs are not in a floodplain and are above the PMF elevation. Therefore, they are not required to consider flood design loads.

3.2.3 Seismic Design

The design of SSCs classified as ITS consider loadings based on the WCS CISF design basis ground motion, which was determined by a probabilistic seismic hazard analysis (PSHA) as discussed in Section 2.6. Probabilistic analysis does not result in the determination of a unique Design Earthquake, such as is the case for a deterministic analysis. Instead, several scenarios and models are used to estimate the likelihood of earthquake ground motions at a site and systematically take into account uncertainties that exist in various hazard parameters. The outcomes are in the form of hazard curves that show the mean annual probabilities or frequencies with which various levels of fault displacement and ground motion are expected to be exceeded.

3.2.3.1 Input Criteria

Andrews County is located within the Southern Great Plains physiographic and tectonic province. As described in Section 2.6, a PSHA was performed to establish the appropriate seismic design basis for the facility. A return period of 10,000 years was determined to be appropriate.

Section 2.6.2 documents the evaluation that demonstrates that the ground surface design response spectrum peak horizontal acceleration for 0.01 seconds is 0.25 g and the vertical is 0.175 g.

To estimate ground motions, four Next Generation of Attenuation (NGA)-West2 ground motion prediction models for the western U.S. (WUS) and the EPRI [3-32] models for the central and eastern U.S. (CEUS) were utilized. For the NGA-West2 models, a time-averaged shear wave velocity (VS) in the top 100 ft (VS30) of 760 m/sec was used. The EPRI [3-32] ground motion models are defined for hard rock or a VS30 of 2,830 m/sec and greater. It is unclear whether the site area should be considered a tectonically active region like the WUS or a stable continental region like the CEUS. It may likely be located in a transition between the WUS and CEUS.

3.2.3.3 Design Response Spectra Derivation

The seismic analysis for the CISF swas performed to be consistent with 10 CFR 72.103 [3-23], U.S. Nuclear Regulatory Commission's NUREG- 0800 "Standard Review Plan (SRP) for the Review of Safety Analyses Reports for Nuclear Power Plants: LWR Edition" [3-3] and NUREG/CR-6728 "Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines" [3-25].

3.2.3.4 Design Time History

Consistent with NRC requirements, horizontal and vertical DRS for a 10,000 year return period and associated strain-compatible properties were developed and provided for the SSI analysis. Three three-component sets of time histories were developed through spectral matching. A final report was produced that describes and summarizes the above analyses in Chapter 2, Attachment D. All calculations were performed in accordance with AECOM's NQA-1 Program. Detailed calculations are contained in calculation WCS-12-05-200-001 in Chapter 2 Attachment D.

Design time histories are used to verify all required components are considered acceptable. Chapter 7 includes further details.

3.2.3.5 Use of Equivalent Static Loads

Chapter 7 of the SAR details the load analyses used in the seismic design and analysis.

For the Vertical Storage Systems storage pad *and the NUHOMS® NITS storage pad,* the soil material properties used are the static properties, equal to or lower than the dynamic soil properties and, therefore, conservative for use in an equivalent static analysis. The soil properties used in the equivalent static *analyses* for the Vertical Storage System storage pads *and the NUHOMS® NITS storage pads* are given in Appendix C of [3-33] and are listed in Table 7-38.

The design criteria used for the Canister Transfer System (CTS) is specified in ASME NOG-1, Section 4000 [*3-34*]. All of the load combinations identified in paragraph 4140 have been evaluated. Controlling load combinations have been used to determine component stresses and then are compared to applicable allowable stresses. The sum of simultaneously applied loads (static and dynamic) do not result in stress levels which would cause any permanent deformation, and thus, the CTS fully meets the requirements of ASME NOG-1 [*3-34*].

CHB structural steel components are analyzed and designed *using static* analysis methods for determining forces and moments on structural steel members as a result of applied service loading conditions. Dynamic analysis methods are used for determining structural steel member forces and moments for factored loading conditions where structural components are subjected to seismic loads.

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Seismic analysis information for the NUHOMS® and Vertical Storage System design criteria are fully described in Appendices A.3, B.3, C.3, D.3, E.3, F.3 and G.3.

3.2.3.6 Critical Damping Values

Critical damping values are in accordance with Regulatory Guide 1.61 [3-27] for a SSE.

3.2.3.7 Basis for Site-Development Analysis

Site-specific vibratory ground motion is determined through evaluation of the seismology, geology, and the seismic and geologic history of the site and surrounding region. This information is contained in the site-specific PSHA (Chapter 2, Attachment D).

3.2.3.8 Soil Supported Structures

The soil supported structures that are analyzed for the CISF design basis ground motion are the ITS Storage Pads*, the CTS, and the CHB.*

3.2.3.9 Soil-Structure Interaction

Soil-structure interaction (SSI) is considered in the design of the storage pads and the CTS. *Assessment of the site soil properties and the CHB dynamic response indicates that Soil-Structure Interaction (SSI) effects on the overall seismic response of the CHB are minimal as demonstrated in Section 7.5.3.3.3. During final design of the CHB, SSI analysis will be performed in accordance with ASCE 4-16.* The soil-supported structures requiring SSI are evaluated by considering the properties and effects of the subsurface established during the geotechnical investigation (Chapter 2, Attachment E). Soil boring logs and soil properties of the WCS CISF site are contained in Chapter 2, Attachment E.

3.2.3.10 Seismic-Systems Analysis

3.2.3.10.1 Seismic Analysis Methods

Seismic Analysis for SSCs designated ITS can be found in Chapter 7.

3.2.3.10.2 Natural Frequencies and Response Loads

A modal analysis studies the dynamic properties of structures under vibrational excitation and determines modes of the structure defined by natural frequencies and other factors. Response loads are developed based on the response-spectrum analysis at the appropriate frequencies.

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7.4 Reinforced Concrete Structures – Important To Safety

The NUHOMS® Horizontal Storage Modules (HSMs), NAC VCCs, storage pads for the vertical systems*, and the CHB foundation and floor slab* comprise the only WCS CISF reinforced concrete structures that are ITS. The individual Appendices describing each of the proposed system components provide the structural descriptions and evaluations for each of the selected cask systems. Table 7-2 provides the cross reference to the applicable appendix and section for each canister/storage overpack where the structural evaluation is discussed.

Reinforced structures associated with the CHB are discussed in Sections 7.5.3.2.3 and 7.5.3.5.

7.5 Cask Handling Building

The Cask Handling Building (CHB) is a two-bay ITS - Category *B* steel structure. The CHB is *175* feet by *193* feet and approximately *72* feet tall with rail access to facilitate cask unloading operations, canister transfer operations, and miscellaneous maintenance activities. Figures 1-7 and 1-8 show the general building layout and building cross section. CHB Structural Design is discussed in Section 7.5.3.

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To facilitate rail car unloading activities for NUHOMS® systems, the CHB design incorporates two overhead bridge cranes rated at 130 tons each for lifting loaded transportation casks from the rail car, removal of impact limiters, and shielding, etc.

All transfer operations to move the NUHOMS® System MP187 and MP197HB transportation casks are accomplished with the transportation casks in a horizontal orientation utilizing a bridge crane *with* lifts limited to a maximum height of 80 inches. The vertical systems will utilize the overhead bridge cranes to remove impact limiters and personnel barriers, and the Vertical Cask Transporter (VCT) is used to move the NAC transportation casks from the rail car to the Cask Transfer System (CTS).

The CHB also houses operations involving both a CTS and a VCT in support of unloading transportation casks and transferring canisters from the NAC transportation casks into the storage casks. Both systems are considered ITS, although the VCT transport of a storage cask to the pad has been evaluated for limited lift height drops.

The CTS and VCT are independently designed and analyzed to meet the intent of NUREG-0612 [7-3], "Control of Heavy Loads at Nuclear Power Plants,"

> "To provide adequate measures to minimize the occurrence of the principal causes of load handling accidents and to provide an adequate level of defense-in-depth for handling heavy loads near spent fuel and safe shutdown systems".

Understanding the WCS CISF will not have safe shutdown equipment or spent fuel pools, it is recognized that the canisters loaded with fuel must be safely and securely handled thereby protecting the fuel from damage and protecting the site and surrounding areas from any potential radiological impacts. Even though the potential for a radiological release is very low, the WCS CISF objective is to prevent the occurrence of load handling accidents. Therefore, the licensing basis is to provide handling systems that are robust to failure which makes the likelihood of a load drop event extremely small.

The VCT is not an overhead hoisting system as defined by any ASME Standard, rather it is a mobile hydraulic gantry crane and adheres to applicable ASME B30.1 requirements. The lift links, lifting pins and header beam are designed, load tested and inspected in accordance with the requirements as specified in ANSI N14.6.

7.5.3 Cask Handling Building Structural Design

This section presents the structural description and design criteria*, and analysis* for the WCS CISF Cask Handling Building (CHB). The CHB *structures are designed to meet the applicable requirements for ITS structures in 10 CFR 72.122 as outlined in NUREG-1567 Section 5.4.4*. The CHB is a two bay steel frame structure with metal siding *and roofing designed to provide a weather-protective enclosure for cask handling operations and* to support two overhead cranes used to move transportation casks from the rail car to the transfer vehicle. The CHB *and its foundations are* ITS - Category *B*. The overhead cranes will also be used to remove or install personnel barriers, impact limiters from the transportation casks. All operations to move the NUHOMS® System MP187 and MP197HB transportation casks are accomplished with the transportation casks in a horizontal orientation.

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7.5.3.1 *Descriptions of Systems, Structures, and Components*

Three separate structural systems are included within the CHB structural design, including the steel-framed building itself, the reinforced concrete foundations for the steel building, and the two overhead bridge cranes. Arrangement of the CHB structures and description of each system are provided in the following subsections. Material specifications utilized for the primary structural components of all CHB structures are summarized in Table 15-1.

7.5.3.1.1 Description of CHB Steel Building

As shown in Figures 7-54 through 7-61, the CHB steel building is a braced frame structure with column centerline grid plan dimensions of 175'-0" (north-south) by 193'-0" (east-west) and an eave height 72'-0" above the top of the concrete foundation (Elevation 100'-0" in the figures). The roof is gabled with 1/4-inch per foot slope on each side and peak ridge elevation of 174'-0 1/8". The north-south plan dimension of the building comprises seven equal bays of 25'-0" spacing, with vertically braced interior bays similar to those shown in Figure 7-56 on column lines A, C, F, H, K, and M. The east-west plan dimension comprises two crane bays with 64'-0" spacing between independent crane support columns that are laterally supported by three separate vertically braced frames at column lines A-C, F-H, and K-M (see Figures 7-55 and 7-56). All seven east-west column lines support a primary lateral roof truss system that is tied together with a secondary north-south bridging roof truss system and horizontal roof bracing at the top and bottom truss chord levels. The primary roof trusses vary in depth from 7'-6" at the eave to 9'-6 1/8" at the ridge. The vertical bracing and primary roof truss arrangement is shown in Figure 7-56, with the secondary bridging roof trusses and horizontal roof truss chord bracing shown in Figures 7-60 and 7-61, respectively.

The objective of the CHB analysis and design for tornado missile impacts is to ensure that structural integrity and stability of the primary framing system is maintained. Therefore, only those members critical to lateral and/or vertical stability of the overall structure are required to survive under any potential tornado missile impact scenario, as demonstrated by sufficient code-based capacity to resist the combination of gravity and tornado wind, APC, and impact demands present in the design load combinations. Other members not required to survive tornado missile impact scenarios are identified as sacrificial, or not critical to structural stability. Two categories of sacrificial members are defined: 1) members that do not serve as critical elements of the overall structure primary lateral or vertical load paths and are not required for overall structural stability, such as beams not serving as collectors or struts; and 2) members that are part of the primary lateral or vertical load paths but have redundant counterparts that are assured to survive if the sacrificial member fails. This second category includes several types of horizontal struts, vertical braces, and the center 'zipper' column of each three-column set on the east-west column lines; in each of these cases the redundant framing arrangement provides secondary lateral and/or vertical load paths and stability framing in case of sacrificial member failure.

The design of sacrificial members and their connections does not require the members to remain attached to the structure after impact (i.e., the sacrificial members may themselves become airborne). This is permitted because the safety-related fuel bearing SSCs inside the building have been designed to resist the full spectrum of Regulatory Guide 1.76 tornado missiles representing the range of potential missiles on the plant site. The sacrificial members are considered rigid building debris components as defined in the missile criteria in Regulatory Guide 1.76 [7-35]. Chapter 12 of the appendices (A.12, B.12, etc.) demonstrate that each cask system component is designed and conservatively evaluated for the most severe tornado and missiles anywhere within the United States (Region I as defined in NRC Regulatory Guide 1.76 [7-35]), therefore, the impact of the sacrificial members on the cask systems is bounded.

During detailed design tornado missile impact, evaluations will verify sufficient capacity of all stability-critical (non-sacrificial) members in the absence of the sacrificial members shown to fail under a given postulated tornado missile strike. This includes evaluation of the remaining structure for all gravity and tornado wind pressure/missile impact demands without any stabilization by or load distribution to the failed sacrificial member(s). The complete set of impact locations includes impacts on representative stability-critical members as well as impacts on representative sacrificial members. The latter cases are necessary to evaluate the demands on the surrounding structural elements when the sacrificial member is impacted.

All Indicated Changes are in response to RAI NP-7-12

The framing arrangement shown in Figure 7-56 and utilized on all seven east-west column lines provides lateral system redundancy, distributed lateral stiffness with limited torsional irregularity, and sufficient lateral stiffness to meet drift limitations for bridge crane supporting structures. These design objectives are further achieved via the arrangement of the roof bracing system (i.e., diaphragm); see Figures 7-54, 7-60, and 7-61. As shown, the primary east-west roof trusses are laterally supported by the secondary bridging trusses framed along the full north-south length of the building at the two wind column lines in each crane bay (Column lines D.1, D.2, I.1, and I.2; a typical section at line I.2 is shown in Figure 7-60). Horizontal diagonal roof bracing in the planes of the top and bottom chords is then provided between the primary and secondary trusses to create a continuous roof diaphragm that assures system redundancy by distributing lateral loads among the north-south and east-west braced column lines. The continuous roof diaphragm also limits relative drift of individual vertical frames subjected to localized lateral forces imparted by the cranes.

The bridge crane support system consists of simply-supported runway girders spanning 25 feet between the aforementioned independent crane support columns. As illustrated in Figure 1-7, the crane runways provide crane access to the complete length of the building in the east crane bay, while in the west crane bay the runways span only the four southernmost east-west column lines (from Line 1 to Line 4). Similar to the main building column lines, vertical bracing is provided in two bays of each crane column line (Lines D, E, I, and J); see the typical section shown in Figure 7-59. The runway girders are built-up steel sections with overall depth of 5'-6". At the top girder flange and at Elevation 136'-2", crane runway tie-back elements are provided to transfer lateral loads from the runway girders to the supporting vertically braced frames. The tie-back elements and their connections are detailed to accommodate flexural displacements of the runway without experiencing fatigue. The crane rail supported by the runway girders is 175 lb-per yard, ASTM A759 crane rail with rail clips sized and spaced to ensure both the rails and rail clips can withstand lateral crane operating loads as well as seismic loads.

Ordinary Concentrically Braced Frames (OCBFs) are selected as the seismic lateral force resisting system for the CHB in both the north-south and east-west directions, in accordance with ASCE 43-05 Table 4-1. Although ASCE 7-16 is not a governing code for CHB design (see Section 15.2.4), OCBFs are permitted by ASCE 7-16 Table 12.2-1 for buildings of any height in Seismic Design Category C and lower. For the seismic site coefficients given in the project geotechnical report (SAR Attachment E), Seismic Design Category C would apply to the CHB per ASCE 7-16 Section 11.6.

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All vertical braces in the CHB are ASTM A1085 round HSS sections, which are the most efficient sections meeting the seismic ductility and slenderness requirements of AISC 341-16. Vertical braces are arranged in multi-story X configurations in both the north-south and east-west directions, to balance braces in tension and compression under lateral loads and to limit unbalanced forces on intersecting columns and struts. For the east-west braced frames, the three-column arrangement for each of the braced frames illustrated in Figure 7-56 is selected to provide vertical and lateral load path redundancy in the event of column damage due to tornado missile impact. Similarly, redundancy is achieved in the north-south braced frames by providing two bays of multi-story X braces (four vertical brace members per level) in each of the north-south braced frames and redundant longitudinal struts between columns (see Figure 7-58). For this configuration, the loss of an individual brace, or connection thereto, would only reduce the contribution of the given braced frame to the strength of the associated building story by 25%. This will result in no loss in overall structural integrity.

Figure 7-55 through Figure 7-60 illustrate typical member size groups utilized for CHB primary framing. Member size classes utilized for each primary framing member category are also summarized in Table 7-41. Further discussion of the CHB structural steel analysis and design is given in Sections 7.5.3.3 and 7.5.3.4.

7.5.3.1.2 Description of CHB Foundation

The principal safety function of the foundation system for the CHB is to transfer design-basis normal operating and extreme environmental loading demands from the building columns and crane support columns to the supporting soils, while providing sufficient resistance to sliding and overturning. These functions are achieved with a foundation consisting of cast-in-place, reinforced concrete footings and pedestals supporting the CHB column base plates. The use of shallow spread-footing type foundations is in accordance with recommendations in the project geotechnical report (see SAR Attachment E). The general foundation arrangement consists of three continuous strip mat footings running north-south, each supporting one of three column line groups shown in Figure 7-55: Lines A-D, Lines E-I, and Lines J-M. Separate footings are provided for the wind column vertical trusses at the north and south ends of the building. All footings are founded at a minimum depth of 9 feet below grade. This depth is selected to provide sufficient pedestal depth for development of the reinforcement and anchor rods required for resistance of tornado-induced uplift demands on the CHB columns. Excavation to the bearing stratum depth ensures the foundations will bear on competent material below the maximum 6.5-foot depth of loose overburden material encountered in boring activities documented in the project geotechnical report. See Section 7.5.3.3.3 for evaluation of soil-structure interaction effects. Further discussion of CHB foundation analysis and design is given in Section 7.5.3.5.

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The working floor of the CHB is provided by a reinforced concrete slab on grade that is structurally isolated from the CHB foundations and the CTS foundation. The slab is founded on compacted structural fill placed to a sufficient depth to remove loose in-situ materials, in accordance with the project geotechnical report. Thickened reinforced concrete sections are provided for support of the rails and railcars at the south end of the building (see Figure 1-7).

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7.5.3.1.3 *Description of CHB Overhead Cranes*

To facilitate rail car unloading activities for NUHOMS® systems, the CHB design incorporates two overhead bridge cranes rated at 130 tons each for lifting loaded transportation casks from the rail car, removal of impact limiters, and shielding, etc. The vertical systems will utilize the overhead bridge cranes to remove impact limiters and personnel barriers, and the VCT is used to move the NAC transportation casks from the rail car to the CTS.

The two cranes are identical in terms of geometry and configuration, which generally consists of two box-beam bridge girders supporting a top-running trolley. As shown conceptually in Figures 1-8 and 7-56, the bridge girders span 64'-0" between crane runway rails, and a minimum height of 40'-2" is provided from hook to finished floor. Bridge and trolley travel are limited by structural steel end stops installed on the crane runway girders and bridge girders, respectively. The end stops engage bumpers installed on the crane and trolley that are sized and configured to limit impact forces applied to the supporting structure. A minimum of 3 inches of clearance is provided in all directions between crane components and surrounding obstructions in the building, in accordance with ASME NOG-1 and CMAA-70.

The overhead bridge cranes are classified as *ITS including the seismic clips and runway beams and supporting structures, and are* designed in accordance with *NOG-1-2015 [7-70] "Rules for Construction of Overhead Gantry Cranes (Top Running Bridge, Multiple Girder)."* The overhead bridge cranes rails are attached to the CHB structure in a manner that provides adequate assurance that the rails will remain attached to the CHB structure during the above-described seismic event. Seismic clips are provided on the overhead crane bridge trucks and trolley to limit uplift during a seismic event, thereby eliminating the potential for the bridge or trolley to fall onto loaded casks inside the CHB.

Lifts performed by the overhead bridge crane are governed by the guidance of NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants: Resolution of Generic Technical Activity A-36," to minimize the potential for release of radioactive material from a spent fuel cask. NUHOMS® transportation/transfer cask lifts are performed using the overhead bridge crane and the lift height is administratively controlled to ensure that the 80-inch design basis drop accidents previously approved by the NRC remain bounding (Reference WCS CISF SAR Tables A.3-1, B.3-1, C.3-1, and D.3-1). The overhead cranes may be used for miscellaneous lifts that do not involve lifting of loads over loaded transportation or storage casks inside the CHB.

7.5.3.2 Design Criteria

Analysis and design of the CHB structures are governed by nuclear facility codes and standards. NUREG-1567 Section 5.4.4, "Other SSCs Important to Safety," references ANSI/ANS 57.9 and the codes and standards cited therein as the basic references for ISFSI structures important to safety. Although ANSI/ANS 57.9 is no longer maintained as an American National Standard, the principal references it cites for analysis and design of ITS steel and concrete structures are consistent with current codes and standards applicable to safety-related nuclear facilities. As also summarized in Section 15.2.4, the following codes and standards are utilized for the given purposes:

- *ANSI/AISC N690-18, Specification for Safety-Related Steel Structures for Nuclear Facilities. Applicable to definition of steel design load combinations and steel member and connection design requirements. ANSI/AISC 360-16, Specification for Structural Steel Buildings, is the baseline document modified in part by ANSI/AISC N690-18 for application to nuclear facilities.*
- *ANSI/AISC 341-16, Seismic Provisions for Structural Steel Buildings. Applicable to definition of seismic design and detailing requirements for the CHB structural steel seismic lateral force resisting system.*
- *ACI 349-13, Code Requirements for Nuclear Safety-Related Concrete Structures. Applicable to definition of concrete design load combinations and design of reinforced concrete structures and anchorages.*
- *ASCE 43-05, Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities. Applicable to evaluation of seismic demand and capacity of the CHB structures.*
- *ASCE/SEI 4-16, Seismic Analysis of Safety-Related Nuclear Structures. Applicable to seismic analysis procedures for the Cask Handling Building and its foundations.*
- *ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures. Applicable to development of normal operating wind loads, snow and rain loads, and overhead crane operating loads.*
- *ASCE/SEI 7-05, Minimum Design Loads for Buildings and Other Structures. Applicable to transforming tornado wind speed into pressures applicable to the CHB, in accordance with NUREG-0800 Section 3.3.2, Tornado Loads.*
- *ASME NOG-1-2015, Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder). Applicable to analysis and design of the two 130-ton overhead cranes supported by the CHB.*
- *CMAA-70 2015, Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes. Applicable to design of the CHB crane runway system.*

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7.5.3.2.1 Load Definitions

The CHB structure is designed to withstand snow and rain in accordance with the International Building Code. In addition, it is designed to resist failure of structural members under concurrent loading by design-basis tornado winds, atmospheric pressure change (APC), and tornado missiles.

Administrative Controls will be used to mitigate certain impacts of design-basis tornado loading. The transportation cask will not be moved into the building to begin the railcar unloading process unless current and forecasted weather for the approaching eight (8) hours indicate safe weather conditions. Eight hours is the estimated time to move any of the casks from the railcar to a stable configuration within the CHB in which the crane is no longer overhead or adjacent. For the NUHOMS® systems, eight hours bounds the approximate time (6.4 hours for MP187 casks, 4.3 hours for MP197HB casks) from entry of the cask railcar into the CHB, to the point where the cask has been placed on the transfer skid and the overhead crane can be relocated to the south end of the CHB. For the NAC systems, eight hours bounds the approximate time (5.5 hours for NAC-STC casks, 6.5 hours for NAC-UTC casks, and 8 hours for NAC-MAGNATRAN casks) from entry of the cask railcar into the CHB, to placement of the canister on the Canister Transfer Facility pad, at which point the overhead crane will no longer be overhead or adjacent to the cask on the railcar. Estimated time to perform cask receipt and transfer activities are provided as occupancy times in the occupational collective dose tables in each cask model's respective Appendix, refer to Tables A.9-2, B.9-2, C.9-2, D.9-2, E.9-1, F.9-1, and G.9-1. Administrative controls will restrict the movement of the overhead crane such that it will remain in the south-most bay of the CHB once railcar unloading has been completed. Administrative controls will prohibit additional non-empty casks on railcars inside the CHB, and thus adjacent to the crane, until the previous cask has been removed from the CHB and the next unloading evolution can proceed, weather conditions permitting. Similarly, for railcar loading operations following retrieval of a loaded canister, the loading process will not be permitted to proceed unless current and forecasted weather for the approaching eight hours indicate safe weather conditions. These actions eliminate the potential for collapse of overhead cranes onto canisters during receipt, transfer, and retrieval operations (with storage operations occurring outside the CHB).

A safe condition and forecast is considered to be the absence of: Tornado and Severe Thunderstorm Watches, Tornado and Severe Thunderstorm Warnings, and predicted wind speeds that would qualify for a Severe Thunderstorm Watch (58 mph or greater). Weather forecasts will be accessed from the NOAA Weather Forecast Office prior to each railcar loading/unloading. The nearest NOAA Weather Forecast Office to the CISF is the Midland/Odessa Office. Administrative controls triggered by the presence of Tornado and Severe Thunderstorm Watches, Tornado and Severe Thunderstorm Warnings, and predicted wind speeds that would qualify for a Severe Thunderstorm Watch ensure avoidance of atmospheric conditions which are favorable for the development of severe thunderstorms capable of producing tornados within the following eight hours.

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This section describes loads, loading combinations and analysis methods to be met for design of the WCS CISF reinforced concrete and structural steel structures.

Loads

Loads used in analysis and design of CHB structure include the following:

- D Dead load
- L Live load
- *C – Crane operating and lifted (hoist) loads*
- *S – Snow load*
- H lateral soil pressure load
- T_o Thermal load
- W Wind load
- *W_t Tornado load*
- F' Flood load
- E' Design Basis Earthquake seismic load

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Load Definitions

- **Dead Load *(D)*** – Defined as any load, including related internal moments and forces, that is constant in magnitude, orientation, and point of application. Dead loads include the mass of the structure, and any permanent equipment loads *including the overhead crane bridge and trolley weights*. *A* minimum uniform load allowance of 20 lb/ft^2 is applied to roof *and elevated platform* areas to account for miscellaneous electrical conduits, handrails and ladders for which the actual dead load contribution is not precisely known at the time the analysis or design is performed.
- **Live Load *(L)*** – Defined as any normal load, including related internal moments and forces that may vary with intensity, orientation and/or location of application. Movable equipment loads, *other than crane loads,* loads due to vibration and any support movement effects and operating load are types of live loads. The following descriptions provide design requirements for various types of live loads.

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  - **Transportation Vehicle Loads and Heavy Floor Loads** – Loads due to vehicular truck and rail traffic in designated building areas are in accordance with standard loadings defined by the American Association of State Highway and Transportation Officials (AASHTO) and by the American Railway Engineers Association. Special heavy loading conditions resulting from transport of SNF and storage casks on truck and rail transporters/carriages are considered. Design basis cask weights bound the worst-case condition of all vendor designs handled at the WCS CISF. Floor loadings from transportation, transfer and storage mode casks are also considered, along with sufficient allowance for any impact resulting from placing the moving loads on the floor or other areas of the structure. Within the building, the floor under the Canister Transfer System will be designed to handle the specific loads produced by the hydraulic gantry system.

- **Floor Live Loads** – A floor live load of 300 lb/ft^2 is applied in areas of heavy *equipment* operation in the CHB. *Live load for stairs, walkways, and platforms is 100 lb/ft^2.*

- ***Crane and Hoist Loads (C)*** *– Design loads for the CHB permanently installed cranes and hoists envelop the full rated capacity of the cranes, including allowances for impact loads and test load requirements. The rated capacity of each of the two overhead bridge cranes in the CHB is 130 tons. Crane test loads are considered in the design at 125% of the rated capacity of the cranes, increased by an additional 5% in accordance with ASME NOG-1-2015 Section 7423. Forces induced by crane movement are calculated in accordance with ASCE 7-16, as follows:*
  - *Vertical impact: 25% of maximum wheel loads (including lifted load and crane self-weight).*
  - *Lateral side thrust: 20% of the sum of the rated hoist capacity, plus the weight of the crane trolley and hoist.*
  - *Longitudinal traction: 10% of maximum wheel loads (including lifted load and crane self-weight).*

- ***Snow Load (S)*** *– As described in Chapter 3, the design live load due to rain, snow, and ice is 10 lb/ft^2, which is the ground snow load. Determination of roof snow and ice loads is in accordance with the requirements of ASCE 7-16.*

- **Hydrostatic Fluid Pressure Loads** – Are due to fluids held in internal building compartments, such as tanks. There are no reinforced concrete tanks in the *CHB*. All tanks located in the *CHB* are designed in accordance with mechanical equipment design criteria.

- **Soil Load (H)** – Based on the density of the soil and includes the effects of groundwater, see attachment E of the WCS CISF SAR Chapter 2. Since the WCS CISF site is a dry, relatively flat site and the CHB is a slab-on-grade structure, no groundwater or soil pressure loads are exerted on building structures. Therefore, determination of lateral soil pressure loads is not necessary for structural analysis or design.

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- ***Thermal Load (T_o)*** *– Consists of thermally induced forces and moments resulting from operation and environmental conditions affecting the CHB. The design temperature changes (ΔT) used for structural analysis and design of the CHB are the differences between expected construction temperatures and winter or summer operating temperatures, assuming the building is unheated and without air conditioning. The temperatures considered for these ΔT calculations are based on data for Midland, Texas in Technical Report No. 65, Expansion Joints in Buildings, which include a 66°F mean temperature during construction, a summer operating temperature of 100°F (exceeded, on average, only 1% of the time between June and September), and a winter operating temperature of 19°F (exceeded, on average, 99% of the time between December and February). This results in a positive ΔT of 34°F and a negative ΔT of 47°F for consideration in the CHB analysis. In accordance with NUREG-1536 and ANSI/ANS 57.9, thermal loads are not combined with tornado or seismic loads given that the CHB thermal loading is self-limiting and will be relieved during response of the structure to these extreme loading conditions.*

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- ***Wind Loads (W)*** *– Are those pressure loads generated by the design (or "normal") wind. The basic wind speed used to determine design wind loads on the CHB walls and roof is 116 miles per hour. Design wind loads are determined in accordance with the requirements of ASCE 7-16 [7-69], which consider ultimate strength level (limit state) wind speeds rather than service level wind speeds. The resulting pressures are intended for use with unity (1.0) LRFD wind load factors in the steel and concrete design load combinations. Wind loading conditions applicable to the CHB Main Wind Force Resisting System are determined in accordance with the Directional Procedure given in ASCE 7-16, Chapter 27 Part 1. Internal pressure coefficients are based upon an enclosed structure, given use of rated doors and operational protocols to shut all CHB doors during inclement weather. Design velocity pressures (q_z) are determined using ASCE 7-16 Equation 26.10-1:*

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$$

where:

K_z = velocity pressure exposure coefficient, equal to 1.18 for Exposure Category C and eave height of 73 feet above ground

K_{zt} = topographic factor, taken as 1.0

K_d = wind directionality factor, equal to 0.85 for Building Main Wind Force Resisting System

K_e = ground elevation factor, taken as 0.9 for site elevation of approximately 3500 feet

V = basic wind speed, equal to 116 mph for the WCS CISF site.

Per NRC RG 1.76, the automobile missile impact is applicable to framing members over all heights from grade to 30 feet above all grade levels within 0.5 miles of the CHB. Based on the stated automobile parking administrative control and minimal elevation changes at the WCS CISF site, the lower 30 feet of primary framing members are considered subject to missile impact. A representative set of all potential strike angles on external framing members is evaluated. Internal primary framing impacts (e.g., crane support columns) afforded by the 25-foot north-south column spacing are also evaluated.

A linear elastic analysis and design approach is taken for missile impact loading on framing members, such that calculated demands can be superimposed on those due to tornado wind, atmospheric pressure change, and other normal loading conditions. The impulsive force magnitude of the automobile traveling at the prescribed velocity is determined using an impulse-momentum procedure. The magnitudes of demands induced in the impacted framing members are a function of both the impulsive force magnitude and the dynamic behavior of the impacted structure. Therefore, for each potential impact location considered, the impulsive force is applied to the structural analysis model as a rectangular step-function load in a transient dynamic analysis. The peak structural demands resulting from these analyses are then superimposed upon those due to tornado wind, atmospheric pressure change, and gravity load cases, in accordance with the design load combinations. Design of CHB primary framing members for these load combination demands ensures that neither the building nor the crane runway support structures will fail under design basis tornado loading, thereby eliminating the potential for damage to canisters during receipt, transfer, and retrieval operations (with storage operations occurring outside the CHB).

For further discussion of tornado missile impact analysis, see Section 7.5.3.3.4.

- **Flood Loads (F')** – Are due to exterior flood waters from the design-basis flood exerting forces and moments on exterior buildings structures, or entering a building and exerting loads on interior building structures. As described in Chapter 2, the CHB finished floor elevation is above the PMF elevation and flood loads are not applicable.
- **Seismic Loads (E')** – *Loads are determined using nuclear facility standards, including ASCE 4-16 [7-69] and ASCE 43-05 [7-44]. In accordance with seismic analysis requirements in these codes, modal response spectrum analysis is performed to determine seismic demands for structural design of the CHB. The input response spectra for this analysis are developed from the site-specific response spectra generated by the Probabilistic Seismic Hazard Analysis for the WCS CISF site (discussed in WCS CISF SAR Chapter 2). Design spectral response accelerations will be used in the analysis and design of the building structure, crane supports, and seismic clips used as restraint for the overhead bridge crane and trolley.*

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Assessment of the site soil properties and the CHB dynamic response indicates that Soil-Structure Interaction (SSI) effects are minimal, such that the criteria of ASCE 4-16, Section 5.1.1 can be applied to justify fixed-base analysis in lieu of detailed SSI analysis. Section 5.1.1(a) permits seismic response analysis without consideration of soil-structure interaction (i.e., fixed-base analysis) if the frequencies of a rigid structure supported on soil springs representing site-specific soil properties are more than twice the dominant frequencies of the actual structure. This condition is present for the dominant lateral, rocking, and torsional response frequencies of the CHB, given the stiff soils at the WCS CISF site. Therefore, the design of the CHB for seismic loading presented herein, is performed using fixed-base analysis utilizing the surface Design Response Spectra (DRS) developed in the Probabilistic Seismic Hazard Analysis for the WCS CISF (discussed in SAR Chapter 2).

It is noted that the ratio of soil-supported to fixed-base response frequencies for vertical response of the crane support system is less than 2 (See Table 7-42). Although this is not a dominant mode with respect to the overall structure (mass participation is less than 10%), the result indicates amplification of the vertical runway response due to SSI effects may occur. Detailed design of the CHB will consider an SSI analysis performed in accordance with ASCE 4-16 [7-71]. The current design of the runway girder presented herein using the fixed base analysis results retains a low maximum demand/capacity ratio (0.29; see Table 7-43) to accommodate potential increased seismic demands during detailed design. The crane-level ISRS utilized for detailed crane design will be generated from the results of SSI analysis as discussed in Section 7.5.3.3.3.

Fixed-base analysis considered for the CHB design is further justified by the separation between the frequency range of the amplified portion of the DRS (approximately 6-20 Hz) and the dominant structural frequencies (less than 4 Hz). ASCE 4-16, Sections 5.1(b) and C5.1.1 indicate that this assessment is a prerequisite for considering a fixed-base analysis in accordance with Section 5.1.1. Regarding the additional fixed-base analysis criteria in ASCE 4-16, Section 5.1.1(b) related to embedment effects, the CHB will be founded on shallow mat foundations in accordance with the geotechnical report recommendations (SAR Attachment E), such that embedment effects will not be significant. Finally, the criterion in ASCE 4-16, Section 5.1.1(c), which requires SSI analysis in all cases where wave incoherency effects are to be considered, is not applicable to the CHB analysis. In accordance with the provisions in ASCE 4-16, Section 5.1.10, ground motion incoherency is conservatively neglected for WCS CISF structures.

For further discussion of CHB seismic load development, see Sections 7.5.3.3.3 (steel building) and 7.5.3.6 (overhead cranes).

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7.5.3.2.2 Structural Steel Load Combinations

Structural steel load combinations applicable to the CHB are based on the LRFD load combinations given in ANSI/AISC N690-18, with the following three basic assumptions:

1. *The design-basis seismic load case discussed above (E) is utilized where the safe-shutdown earthquake load (SSE) appears in the ANSI/AISC N690-18 load combinations. Load combinations with operating-basis earthquake loads applicable to nuclear power plant SSCs are not applicable to CHB design.*

2. *As previously stated, self-limiting operating thermal loads are not combined with tornado or seismic loads, in accordance with ANSI/ANS 57.9.*

3. *Since wind loads are developed per ASCE 7-16 using ultimate wind speeds, use of a 1.0 load factor on the wind load case (W) is appropriate in the severe environmental load combinations.*

4. *Crane load (C) is included with normal wind load (W) and seismic load, but is neglected with tornado loads (W_t) given the aforementioned crane administrative controls for tornado warnings. This is in accordance with ANSI/AISC N690-18 Equations NB2-4 and NB2-7.*

5. *For uplift load combinations, 90% of dead load is considered in conjunction with 100% of operating crane loads with a destabilizing effect (i.e., crane vertical impact, side thrust, and longitudinal traction loads). This is in accordance with ANSI/AISC N690-18 Section NB2.5d(4).*

The following are structural steel design load combinations that result from these assumptions, when reduced to contain only the load cases previously defined as applicable to the CHB:

1. $1.4D + C + T_o$

2. $1.2D + 1.6L + 1.4C + 0.5S + 1.2T_o$

3. $1.2D + 0.8L + 1.4C + 1.6S + 1.2T_o$

4. $1.2D + W + 0.8L + C + 0.5S + T_o$

5. $D + 0.8L + C + E$

6. $D + 0.8L + W_t$

7. $0.9D + C + W$

8. $0.9D + C + E$

9. $0.9D + W_t$

7.5.3.2.4 Overhead Crane Load Combinations

Crane Load combinations applicable to the design of the overhead bridge cranes are developed in accordance with ASME NOG-1 Section 4140. The design-basis seismic load (E) discussed above is considered in the safe-shutdown earthquake (SSE) load case in the ASME NOG-1 extreme environmental load combinations.

7.5.3.3 CHB Steel Building Structural Analysis

To evaluate the performance of the CHB steel framing shown in Figures 7-54 through 7-61, the building is modeled in a detailed three-dimensional structural analysis model and subjected to all of the applicable design load cases and load combinations defined above in Sections 7.5.3.2.1 and 7.5.3.2.2. The assumption of linear elastic response for static, seismic, and tornado wind loads permits separate analysis of each loading condition and superposition of applicable load case member forces and moments to determine total load combination demands for evaluation vs. code defined member capacities.

In accordance with ANSI/AISC 360-16 Chapter C (as referenced by ANSI/AISC N690-18 Chapter NC), the First-Order Analysis Method is used to address stability analysis requirements. The CHB meets AISC limitations for use of this method, since the lateral system consists of a highly redundant braced frame with minimal second-order deformations (P-Δ). This method is also considered the most appropriate approach for dynamic analysis of the CHB. The member stiffness reductions required by other stability methods, such as the Direct Analysis Method, would result in unrealistic modal responses for the CHB braced frames, as the columns and struts are expected to remain elastic under design basis seismic loading. In addition, the Direct Analysis Method requires second-order, nonlinear analysis, which is not compatible with the modal response superposition performed in both the CHB seismic and tornado missile analyses.

7.5.3.3.1 CHB Steel Building Structural Analysis Model

Figure 7-62 shows an isometric view of the three-dimensional finite element analysis model generated in program STAAD.Pro (STAAD). The STAAD version utilized is the CONNECT Edition, Version 22.01.00.38, which is verified and validated under an ASME NQA-1 compliant quality program.

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The global coordinate system for the CHB STAAD model is defined with positive X eastward, positive Y upward, and positive Z southward. The global boundary conditions modeled in all static and dynamic loading cases in STAAD consist of pinned supports at the base of each column. Each pinned base restrains the global UX, UY, and UZ translations, as well as ROTY rotations for analysis stability. The pinned base nodes are modeled at the bottom of column base plate elevation. Local boundary conditions applicable to individual members typically involve pinned member end releases (local ROTY and ROTZ) for all beams, vertical braces, and horizontal braces, as well as at the top of columns where they connect to the continuous roof truss chords.

The model includes approximately 3100 nodes and 5800 beam elements, with the intent of sufficient refinement to provide an accurate assessment of structure response to static and dynamic loading. The STAAD beam elements are formulated with six degrees of freedom per node (three translations and three rotations) and with shear deformation effects included in the member stiffness matrix. STAAD utilizes a diagonal, lumped mass matrix approach, with mass terms at all active degrees of freedom. Since dynamic analysis is performed to evaluate the CHB for seismic and tornado missile loading, members with significant transverse loading between points of support (e.g., beams and girders) are subdivided into multiple beam elements to capture dynamic flexural responses while utilizing the STAAD lumped mass formulation. At a minimum, three intermediate nodes (four elements) are used for all beams and girders.

Member stiffness properties for all rolled shapes are assigned using built-in AISC section property tables provided in STAAD, while properties for built-up sections such as the crane runway girders are manually calculated and inputted. Bridge crane and trolley members are not modeled in the CHB STAAD model; rather, the mass of the bridge is proportionally distributed to the runway girders while the trolley and lifted load mass is distributed to the runways according to trolley position along the bridge. Other entities modeled only as applied mass include secondary framing members and elements, such as girts, purlins, siding, roofing, and floor deck.

Linear elastic, isotropic material properties are assigned for all steel members in the CHB analysis model, including elastic modulus (E), Poisson's ratio (ν), unit weight (γ), and coefficient of thermal expansion (α). See Table 15-2 for the material property values utilized.

7.5.3.3.2 Static Analysis

Static analyses are performed to determine member forces, column reactions, and structure deflections due to gravity loads, crane operating loads, and wind/tornado pressures. The overall dead (D), crane (C), wind (W), and tornado wind (W_t) load cases defined in Section 7.5.3.2 are subdivided into several separate static load cases as needed to develop design load combinations that include enveloping directional permutations. Separate static load cases are modeled and analyzed for structure dead load, live load, crane dead load, crane lifted load, and crane impact loads in each direction (vertical, lateral, and longitudinal). With regard to wind load (W), separate static load cases are modeled for each primary direction of wind loading (i.e., +X, -X, +Z, and –Z), each containing the associated windward, leeward, sidewall, and roof pressures. Internal pressures are also addressed in a separate static load case. These are then combined in accordance with the ASCE7-16 Directional Procedure, as discussed in Section 7.5.3.2. A similar approach is used for tornado wind pressures, with a separate static load case for each primary direction of wind pressure loads (W_w) and for atmospheric pressure change (W_p).

Static analysis is also performed for the operating thermal (T_o) load case to evaluate forces induced in the CHB due to restraint of building temperature changes between ambient construction and winter or summer operating temperatures, as discussed in Section 7.5.3.2.1. Two load cases are developed to apply uniform temperature changes (ΔT) to all CHB framing equal to +34°F and -47°F, as previously defined. In accordance with ANSI/ANS 57.9, the resulting forces and moments are combined with gravity load cases within normal operating load combinations, but are not applied for extreme environmental conditions.

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7.5.3.3.3 Seismic Analysis

The seismic response of the CHB is evaluated using modal response spectrum analysis, in accordance with ASCE 43-05 and ASCE 4-16. The input response spectra for the analysis are developed from the site-specific response spectra generated by the PSHA for the WCS CISF site (discussed in SAR Chapter 2).

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Evaluation of Soil Structure Interaction Effects

Per ASCE 43-05 [7-44] Section 3.1 and ASCE 4-16 [7-71] Section 5.1(a), soil-structure interaction (SSI) effects must be considered. To evaluate the significance of SSI effects for the CHB, an assessment of site soil properties and dominant structural frequencies is performed in accordance with ASCE 4-16 Section 5.1.1. This evaluation entails calculation of soil frequencies based on a single degree-of-freedom system consisting of the lateral, vertical, torsional, or rocking soil spring and the relevant mass or mass moment of inertia for the overall CHB. The mass of the embedded CHB foundation is neglected in this calculation. Equivalent soil spring stiffness terms are calculated in accordance with ASCE 4-16 Table 5-2, using strain-compatible shear modulus determined from the site PSHA. In accordance with report NIST GCR 12-917-21 [7-72], soil shear modulus was averaged over an effective depth equal to half the effective footing width for the given direction of motion. Equivalent rectangular foundation dimensions for horizontal, rocking, and torsional responses are calculated on the basis of the combined contact areas of the three primary strip mat foundations as preliminarily sized. For vertical response, the dimensions of a single strip mat are considered, since the most significant mode involves vertical response of the crane runway support systems supported on the center strip mat.

As shown in Table 7-42, all soil/structure frequency ratios for lateral responses, torsion, and rocking in the east-west direction exceed 2. The ratio for rocking in the north-south direction is very nearly 2 (1.98). These results indicate that evaluation of overall structural response of the CHB to seismic loading based on a fixed-base analysis is acceptable with respect to ASCE 4-16 Section 5.1.1. However, the ratio of the rigid/soil-supported frequency to fixed-base structure frequency for vertical response of the crane support system is substantially less than 2 (1.64 for analysis including 130-ton cranes). Although the vertical response frequencies considered for these ratios are not associated with dominant modes involving overall structure response (the modes have small overall mass participation of approximately 10% in the vertical direction), the given frequency ratios indicate that some amplification due to soil-structure interaction effects may occur in the crane support system vertical seismic response. For the design presented herein, considering fixed-base analysis, the demand/capacity ratios of the crane runway girder are held lower to accommodate potentially larger seismic demands (maximum calculated DCR is currently 0.29; see Table 7-43). Detailed design of the CHB will consider an SSI analysis performed in accordance with ASCE 4-16. Performing an SSI analysis will ensure amplification due to SSI effects is captured in the crane support system response and in the resulting crane ISRS needed for detailed design of an ASME NOG-1 crane.

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Use of a fixed-base analysis for this stage of the CHB seismic design presented herein is further justified by the relatively low Demand/Capacity Ratios (DCRs) calculated for the selected member sizes under seismic load combinations. As shown in Table 7-43, tornado wind/missile load combinations govern the design for the vast majority of CHB framing member types. The tornado loading DCRs for CHB primary framing elements, such as columns, roof truss chords, and vertical bracing, are more than twice the corresponding seismic DCRs. Thus, any small increases in seismic demands that may result from SSI analysis performed in final design are not expected to require changes to the CHB primary framing design. With regard to the runway girder design, the governing DCR in the current design has been held to a low value (0.29 from Table 7-43) to accommodate final design, including potentially increased seismic demands resulting from SSI analysis performed in final design.

Seismic Mass

In accordance with ASCE 43-05 Section 3.4.2, the effective seismic mass of the CHB is taken as the sum of the weight of the structure, permanent equipment, and the expected live load, taken as 25% of the specified design live loads. Also per ASCE 43-05 Section 3.4.2, snow load need not be included in seismic mass since it is less than 30 psf (10 psf is the ground snow load; see Section 7.5.3.2.1). The overhead crane bridge and trolley mass are included for seismic mass in all directions, while the lifted load mass is only considered as seismic mass in the vertical direction. This is based on the assumption that the pendulum motion of the lifted load is of sufficiently low frequency to be considered as fully out-of-phase with the dynamic response of the supporting structure.

Damping

A constant modal damping ratio of 7% is used for CHB seismic analysis. This is based on ASCE 43-05 Table 3-2, considering welded or friction-bolted structures at Response Level 3. In accordance with AISC 341-16 Section D.2.2, all OCBF bolted connections will be classified as friction-bolted, as they are required to utilize fully pretensioned bolts with faying surfaces prepared to a slip coefficient of Class A or better. Per ASCE 43-05 Section 3.4.3, Response Level 3 damping may be used for evaluating seismic-induced forces and moments in structural members by elastic analysis, without consideration of the actual response level for Limit States A, B, or C. The CHB analysis considers Limit State C, corresponding to limited permanent distortion per ASCE 43-05 Table 1-4.

Modal Analysis

Modal response of the CHB for seismic response spectrum analysis is evaluated in STAAD using the Load-Dependent Ritz eigensolver. This solver is used because it is more efficient than other solvers in extracting modes of significance to the seismic response of the building. As a result, fewer overall modes are required to obtain sufficient mass participation. 500 modes are extracted, capturing more than 90% mass participation in all three global directions.

Response Spectrum Analysis Methodology

Response spectrum analyses for the CHB are performed using the Lindley-Yow method described in NRC Regulatory Guide 1.92 and endorsed by ASCE 4-16 Section 4.3.2. The Lindley-Yow method divides the total seismic response into two components: response in-phase with the ground motion (i.e., the "rigid" response) and response out-of-phase with the ground motion (i.e., the "periodic" response). A typical seismic response spectrum can be divided into three regions, as shown in Figure 7-63. Defining f_{SP} as the frequency corresponding to the peak spectral value on the response spectrum curve and f_{ZPA} as the frequency corresponding to the zero-period ground acceleration (ZPA), the regions may be categorized as follows:

- *Modes having a frequency less than f_{SP} (low-frequency range) are predominately out-of-phase with the ground motion and thus have no contribution to the in-phase response.*
- *Modes having a frequency between f_{SP} and f_{ZPA} (mid-frequency range) contribute to both the in-phase and out-of-phase responses.*
- *Modes having a frequency greater than f_{ZPA} (high-frequency range) are in-phase with the ground motion.*

The total in-phase response is calculated using the "Static ZPA Method" outlined in Regulatory Guide 1.92. This involves a static analysis in which the ZPA is applied to the total in-phase mass, equal to the total structure mass minus the sum of modal masses for modes with $f < f_{sp}$. Applying the ZPA to the in-phase mass automatically accounts for the so-called "missing mass," or that portion of the structural mass that does not participate in the amplified modal responses.

The out-of-phase response is determined by performing a response spectrum analysis combining the response of modes having a frequency less than or equal to the frequency corresponding to the ZPA (f_{ZPA}). Modified spectral accelerations, $S'_{ai} = S_{ai} [1- \alpha_i^2]^{1/2}$ are used in the analysis, where S_{ai} equals the unmodified spectral acceleration for mode "i". For modes that have a frequency less than f_{SP} (low-frequency range) and are predominately out-of-phase with the ground motion, $\alpha i = 0$. For modes having a frequency between f_{SP} and f_{ZPA} (mid-frequency range), $\alpha_i = ZPA/S_{ai}$.

Modal responses obtained using the modified spectra are combined in STAAD using the Complete Quadratic Combination (CQC) method, in accordance with ASCE 4-16 Section 4.3.2. The total seismic response in each direction is calculated as the square root of the sum of the squares (SRSS) of the in-phase and out-of-phase components, in accordance with the Lindley-Yow Method outlined in Regulatory Guide 1.92. Finally, the three directional responses (or spatial components) are combined by SRSS, in accordance with ASCE 4-16 Section 4.3.3.

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Accidental Torsion

In accordance with ASCE 4-16 Section 3.1(i), the effect of accidental torsion is addressed in static analysis considering a torsional moment equal to the story shear at each level multiplied by 5% of the plan dimension perpendicular to the direction of motion. Two accidental torsion load cases are defined; one involving all story shears in the X direction with corresponding Z-direction eccentricity, and one with all story shears in the Z direction and corresponding X-direction eccentricity. In accordance with ASCE 4-16, the resulting forces must only be used to increase member design forces. Therefore, the magnitudes of the demands calculated in these two load cases are added to the corresponding demands obtained from the response spectrum analysis, which do not have signs as a result of CQC and SRSS squaring procedures.

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7.5.3.3.4 Tornado Missile Impact Analysis

Refer to the discussion of Tornado Loads in Section 7.5.3.2.1 for an introduction to the Tornado Missile Impact Analysis.

The transient dynamic analysis performed in STAAD utilizes the mode superposition method of calculating structural response at each time step. Similar to the seismic response spectrum analysis, the Load-Dependent Ritz eigensolver is utilized, as it is more effective in capturing high frequency modes important to tornado missile response. A sufficient number of modes are extracted to capture more than 90% mass participation. A time step of 0.0001 seconds is considered for the transient analysis, which is well less than 1/20th of the shortest structural response period of interest, in accordance with industry practice. A constant modal damping ratio of 5% is assumed. The impulsive missile loading for the given impact location is applied as a nodal load with a rectangular load vs. time function that has a magnitude equal to that of the calculated impulsive force and a duration of 0.05 seconds. This duration is in accordance with guidance on automobile tornado missile impacts in UCRL-ID-115234, Title I Wind/Tornado Design Guidelines for New Production Reactors," Lawrence Livermore National Laboratory, September 1993. As maximum member forces are shown to occur within the first second of dynamic response, the total duration of the transient analysis is two seconds.

For each impact location of interest, a separate STAAD model is executed to perform static analyses for all other tornado wind, APC, and gravity load cases in the tornado load combinations, along with the mode superposition transient analysis for the single automobile impact case under consideration. Member demands are calculated in accordance with the design load combinations for each tornado missile impact model for all primary framing members in the STAAD model, and the envelope of all load combination demands from all models are considered in the member design checks.

7.5.3.4 CHB Steel Building Design

Design of the CHB steel framing is performed in accordance with the requirements of ANSI/AISC N690-18, which overlays additional requirements on the provisions of ANSI/AISC 360-16. This is in general accordance with the NUREG-1567 reference to ANSI/ANS 57.9, which in turn references ANSI/AISC N690-1984 for steel structure load combinations and design limits. ANSI/AISC N690 is considered for CHB design because it provides specific requirements for safety-related nuclear structures, including load combinations containing tornado loading. The 2018 version is utilized for compatibility with current national consensus codes and standards providing requirements for building structures (e.g., IBC 2016 and ASCE 7-16).

With regard to seismic design, the CHB lateral force resisting system is evaluated in accordance with the design requirements and acceptance criteria given in ASCE 43-05. ASCE 43-05 identifies OCBFs as acceptable structural systems for use in nuclear facilities, and permits design of steel structures in accordance with LRFD requirements given in AISC specifications (AISC 360 or AISC N690), as modified by the AISC Seismic Provisions (see ASCE 43-05 Section 4.2.4.) Thus, the CHB OCBFs are designed to meet the system, member, and connection requirements given in ANSI/AISC 341-16, Section F1.

Both ASCE 43-05 and ANSI/AISC 341-16 ensure acceptable seismic performance of OCBF systems by requiring design of critical members and connections for larger seismic demands than those considered for vertical brace member design. In the design of the CHB OCBFs in accordance with ASCE 43-05, the full seismic force developed from the elastic analysis is considered for design of all members and connections except vertical brace members. The design seismic force for the vertical braces is taken as the elastic seismic demand divided by the specified System Inelastic Energy Absorption Factor ($F_{\mu s}$; see ASCE 43-05 Section 5.1.2). For design of the CHB to Limit State C, the $F_{\mu s}$ factor applicable to OCBF vertical bracing members is 1.5 (see ASCE 43-05 Table 5-1). The CHB has no weak or soft stories and its fundamental frequencies are less than the amplified acceleration region of the design response spectrum; therefore $F_{\mu s} = F_{\mu}$. Thus, design of the CHB per ASCE 43-05 ensures that inelastic response under seismic loading will first occur in the vertical braces, while the columns and beams are designed not to buckle under the design-basis seismic loads (i.e., those calculated in the elastic analysis with $F_{\mu} = 1.0$).

All Indicated Changes are in response to RAI NP-7-12

7.5.3.4.1 Member Design

Design of the CHB structural steel framing confirms that no applicable strength or serviceability limit state is exceeded when the structure is subjected to the design load combinations. In terms of strength limit states, the design compares all individual and combined loading member demands calculated from the design load combinations evaluated in the STAAD analysis model with the corresponding LRFD design strengths. In accordance with ANSI/AISC N690-18, member design strengths are calculated per ANSI/AISC 360-16 Chapters D through H, without modification. In general, the design for each member and each applicable strength limit state confirms:

$$R_u \leq \phi R_n$$

where R_u is the required strength (load combination demand), R_n is the nominal strength, and ϕ is the applicable resistance factor defined in ANSI/AISC 360-16.

With regard to serviceability, seismic story drifts are confirmed to meet the drift ratio limit specified in ASCE 43-05 for concentrically braced frames designed to Limit State C, which is 0.005. Additionally, the crane runway girders are confirmed to have lateral and vertical deflections less than the serviceability limits specified in CMAA-70 (L/400 for lateral deflection and L/600 for vertical deflection) under service level loading conditions.

STAAD Code Checking

Member strength design checking is performed in accordance with ANSI/AISC 360-16 LRFD provisions using the code checking capabilities provided in STAAD. Code checks are executed for all analyzed members and all design load combination demands calculated in each STAAD analysis model. This includes the primary model executed to determine gravity, normal wind, and seismic load combination demands, and separate models executed to determine load combination demands due to the combined effects of tornado wind, APC, and tornado missile impacts at each of the locations considered. Within the primary model used for seismic analysis and design, additional load combinations applicable only to vertical brace member design are defined with seismic load case demands divided by $F_{\mu\sigma}$ = 1.5.

Execution of ANSI/AISC 360-16 code checks within STAAD requires user entry of all applicable member design parameters required for calculation of member design strengths. This includes the specified minimum yield strength of the modeled members, equal to 50 ksi for all CHB members, and various parameters defining the unbraced lengths for each member. Unbraced length parameter inputs include the following:

- *K: Effective length factor, taken as 1.0 for all members in accordance with the First-Order Analysis Method (see AISC 360-10 Appendix 7.3).*
- *LX: Member unbraced length for torsional and flexural torsional buckling.*

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- *LY / LZ: Member unbraced lengths for compression buckling about the member Y and Z axes.*
- *UNT / UNB: Unsupported lengths of member top and bottom flanges in flexural compression, for evaluation of lateral torsional buckling.*

STAAD performs member strength checks for the demands calculated at each end of every member, as well as at 11 equally-spaced points along the member length (1/12th points). The maximum Demand/Capacity Ratio (DCR) for any of these points is presented for each member in the STAAD postprocessor, along with the governing load combination and the governing ANSI/AISC 360-16 strength equation. The governing DCR for each CHB member is taken as the maximum DCR calculated in all STAAD CHB models.

It is noted that STAAD AISC code checking considers the limiting width-to-thickness (member slenderness) ratios defined for members subjected to axial compression and flexure in ANSI/AISC 360-16 Chapter B. However, the seismic ductility and slenderness limits specified in ANSI/AISC 341-16 are not evaluated in STAAD. In accordance with ANSI/AISC 341-16 Section F1.5, all OCBF vertical braces are confirmed in separate calculations to be moderately ductile and to have member slenderness ratios (L/r) less than $4\sqrt{(E/F_y)}$.

7.5.3.4.2 Connection Design

CHB structural steel framing connections utilize shop-welded and field-bolted detailing, to minimize field welding and field weld inspection. Design of CHB framing connections is performed in accordance with ANSI/AISC 360-16 Chapter J, as modified by ANSI/AISC N690 Chapter NJ, and AWS D1.1 and AWS D1.8 where required. The required strengths of connections are determined from all applicable design load combinations, including seismic and tornado load combinations. In addition to meeting the general requirements of ANSI/AISC 360-16, all primary lateral force resisting system connections are designed and detailed in accordance with the provisions applicable to OCBFs in ANSI/AISC 341-16. The following is a summary of applicable requirements implemented in the CHB design:

- *All bolts are high strength bolts installed with full pretension.*
- *Bolts and welds do not share the same force component in any connection.*
- *Bolts are installed in standard holes or in short slots perpendicular to the applied load.*
- *The available shear strength of bolted joints is calculated as that for bearing-type joints in accordance with ANSI/AISC 360-16 Chapter J.*
- *Faying surfaces are prepared to satisfy slip-critical connection requirements in ANSI/AISC 360-16 and are prepared to have a Class A slip coefficient or higher.*

- *The required strength of OCBF vertical brace connections is determined using the overstrength seismic loads, in accordance with AISC 341-16 Section F1.6a. This requirement is met by designing for F_{μ} = 1.0 seismic demands, in accordance with ASCE 43-05.*
- *All OCBF welded connections are detailed and installed in accordance with the applicable requirements of AWS D1.1 and D1.8 as required.*
- *Column base connections and splices are designed for the required axial, shear, and flexural forces defined in ANSI/AISC 341-16 Sections D2.5 and D2.6.*
- *The available strengths of concrete and reinforcing steel utilized in column base anchorage to the foundation are determined in accordance with ACI 349-13.*

7.5.3.5 Reinforced Concrete Structural Analysis and Design

Analysis and design of the CHB reinforced concrete foundations is performed in accordance with the requirements of ACI 349-13, considering all design load combinations defined in Section 7.5.3.2.3. This is in general accordance with the NUREG-1567 reference to ANSI/ANS 57.9, which in turn references ACI 349-85 for concrete load combinations and design limits. Design of CHB column baseplate anchorage is in accordance with the requirements of ACI 349-13 Appendix D.

Material properties considered in foundation analysis and design, including specified strengths for structural concrete, reinforcing steel, anchor rods, and steel plate (utilized for baseplate shear lugs) are summarized in Table 15-2. Soil properties considered in foundation design are those specified in the project geotechnical report (SAR Attachment E). This includes an allowable bearing pressure of 3000 lb/ft^2 and a subgrade modulus of 150 lb/in^3. As stated in the geotechnical report, the allowable bearing pressure is permitted to be increased to 4000 lb/ft^2 (33% increase) for load combinations that include transient loads (such as wind, seismic, and tornado loads). The unit weight of structural fill considered in foundation stability calculations is assumed to be 110 lb/ft^3.

Foundation stability is evaluated for the west strip mat foundation, which is considered representative of all three strip mats. The east and west strip mats have a narrower plan dimension in the east-west direction than the center strip mat, while the west strip mat has somewhat less applied dead load with fewer crane columns than the east strip mat. A minimum factor of safety of 1.5 is required for sliding and overturning when evaluated for the stability load combination containing normal wind and crane operating loads in Section 7.5.3.2.3 (load combination #6). For the seismic and tornado uplift load combinations (#7 and #8 in Section 7.5.3.2.3), the minimum factor of safety for sliding and overturning is 1.1. This is in accordance with ASCE 43-05 Section 7.2 for seismic stability.

All Indicated Changes are in response to RAI NP-7-12

7.5.3.7 Summary of Maximum Design Capacity Ratios

Design Capacity Ratios DCRs are specified for key structural elements of the CHB, which include main columns, sacrificial zipper columns, sacrificial and non-sacrificial struts, built-up crane runway girder, top and bottom roof truss chords, roof truss web members, and sacrificial vertical bracing. The governing DCR for an element group are taken directly from the CHB STAAD model and submodels considering gravity, seismic, tornado wind pressure, and tornado missile impact with tornado wind pressure load combinations. The maximum DCRs for the primary framing structural steel in the CHB STAAD model and submodels are provided in Table 7-43.

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7.5.3.8 On-Site Accidents

WCS CISF-initiated explosions are not considered credible since insufficient explosive materials are present to initiate an event that would result in the destruction of the building. During operations, the amount of flammable liquids that are in the CHB will be administratively controlled to ensure the amount of flammable liquids is maintained below the fire load limits for the respective systems (e.g., 300 gallons of diesel fuel *equivalent* for NUHOMS® *and 50 gallons of diesel fuel equivalent for the NAC-MPC, NAC-UMS, and MAGNASTOR Systems*). In combination with fuel limitations and a fire suppression system, the fire hazard for the building is adequately mitigated (see WCS CISF SAR Section 3.3.6).

7.5.3.9 Off-Site Accidents

Off-site accidents are addressed in WCS CISF SAR Section 12.2.2.

7-58 Nuclear Energy Institute (NEI), "Consistent Site-Response/Soil-Structure Interaction Analysis and Evaluation," June 2009.

7-59 Deleted.

7-60 Deleted.

7-61 ANSI/AISC N690-06, "Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures in Nuclear Facilities."

7-62 ANSI/AISC 360-05, "Specification for Structural Steel Buildings."

7-63 APA Consulting Computer Code SASSI, Version 1.0.

7-64 ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures."

7-65 ANSYS Computer Code and User's Manual, Version 16.0.

7-66 Calculation AREVATN00l-CALC-002, Rev. 0 "Soil Structure Interaction Analysis of TN Independent Spent Fuel Storage Installation (ISFSI) Concrete Pad at Andrews, TX."

7-67 Calculation AREVATN00l-CALC-001, Rev. 1 "ISFSI Pad Design for WCS at Andrews, Texas."

7-68 ACI 349-13, "Code Requirements for Nuclear Safety-Related Concrete Structures and 731Commentary."

7-69 ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures."

7-70 ASME NOG-1-2015, "Rules for Construction of Overhead Gantry Cranes (Top Running Bridge, Multiple Girder)," The American Society of Mechanical Engineers, 2015.

7-71 ASCE/SEI 4-16, "Seismic Analysis of Safety-Related Nuclear Structures," American Society of Civil Engineers, 2016.

7-72 NIST GCR 12-917-21, "Soil-Structure Interaction for Building Structures," September 2012.

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Table 7-41
Cask Handling Building Primary Framing Member Sizes

Structural Element	***Member Size Class***
Main Building Columns	*W14*
Crane Columns	*W14*
Wind Columns	*W14*
Wind Column Vertical Truss Web Members	*2L8x8*
North-South Struts	*W14, W18*
East-West Struts	*W12, W16*
East-West Vertical Braces	*HSS8.625 (round)*
North-South Vertical Braces	*HSS9.625, HSS5.5 (round)*
Intermediate Level Horizontal Braces	*WT*
Primary Roof Truss Chords	*W14*
Secondary Roof Truss Chords	*W14*
Primary Roof Truss Web Diagonal Members	*2L5x5*
Secondary Roof Truss Web Diagonal Members	*2L8x6*
Interior Roof Truss Web Vertical Members	*2L3.5x3.5*
Exterior Roof Truss Web Vertical Members	*W8*
Primary Roof Horizontal Braces	*HSS7x7 (square)*
Secondary Roof Horizontal Braces	*WT*

Table 7-42
Cask Handling Building Evaluation of Soil and Structural Dominant Frequencies

Mode	***Frequency of Rigid Mass on Soil Spring, f_{soil} (Hz)***	***CHB Fixed-Base Dominant Frequency, f_{CHB} (Hz)***	***Ratio f_{soil}/f_{CHB}***
Horizontal, E-W (X)	*16.1*	*3.02*	*5.32*
Horizontal, N-S (Z)	*16.1*	*3.47*	*4.63*
Vertical (Y)	*20.8*	*12.68*	*1.64*
Rocking in E-W direction (about Z)	*10.6*	*3.02*	*3.51*
Rocking in N-S direction (about X)	*6.9*	*3.47*	*1.98*
Torsion	*8.4*	*3.76*	*2.22*

Table 7-43
Maximum Design Capacity Ratios (DCRs)

Element	*Load Combination*				
	Gravity	***Seismic***	***Tornado Wind Pressure[1]***	***Tornado Missile Impact with Tornado Wind Pressure[2]***	***Governing DCR[6]***
Main Column	*0.17*	*0.27*	*0.21*	*0.65*	*0.65*
Zipper Column[3]	*0.08*	*0.10*	*0.15*	*0.16*	*0.16*
Crane Column	*0.30*	*0.28*	*0.11*	*0.65*	*0.65*
Wind Column	*0.12*	*0.12*	*0.13*	*0.70*	*0.70*
Sacrificial Strut[3]	*0.23*	*0.37*	*0.15*	*0.66*	*0.66*
Non-Sacrificial Strut[4]	*0.23*	*0.37*	*0.15*	*0.70*	*0.70*
Crane Girder[5]	*0.19*	*0.29*	*0.05*	*0.11*	*0.29*
Roof truss bottom chord	*0.12*	*0.16*	*0.38*	*0.62*	*0.62*
Roof truss top chord	*0.15*	*0.22*	*0.33*	*0.65*	*0.65*
Roof truss web member	*0.62*	*0.57*	*0.61*	*0.68*	*0.68*
Sacrificial N-S Vertical Bracing[3]	*0.21*	*0.12*	*0.12*	*0.61*	*0.61*
Sacrificial E-W Vertical Bracing[3]	*0.32*	*0.34*	*0.28*	*0.83*	*0.83*
Sacrificial Crane Vertical Bracing[3]	*0.32*	*0.25*	*0.12*	*0.19*	*0.32*

1. *The Tornado Wind Pressure DCRs do not reflect tornado missile impact; i.e., automobile. Columns are generally sized for missile impact.*
2. *Not all possible missile impact locations have been considered in this preliminary analysis. DCRs reflected are based on representative sampling of primary member and framing system impact locations. During detailed design, the governing DCR may increase (see Note 6).*
3. *Sacrificial members hit directly or in close proximity to a tornado missile are allowed to fail. These member DCRs are reflective of an indirect missile strike.*
4. *Non-Sacrificial members are designed to withstand a missile impact. These DCRs are indicative of a member that is directly impacted by a tornado missile. Unless noted otherwise, all members are non-sacrificial.*
5. *The DCRs for the crane girder do not consider all crane position loading scenarios and fatigue to be addressed in detailed design. These considerations may result in an increase in DCR (see Note 6).*
6. *During detailed design, the maximum member DCR shall not exceed 0.90.*

All Indicated Changes are in response to RAI NP-7-12

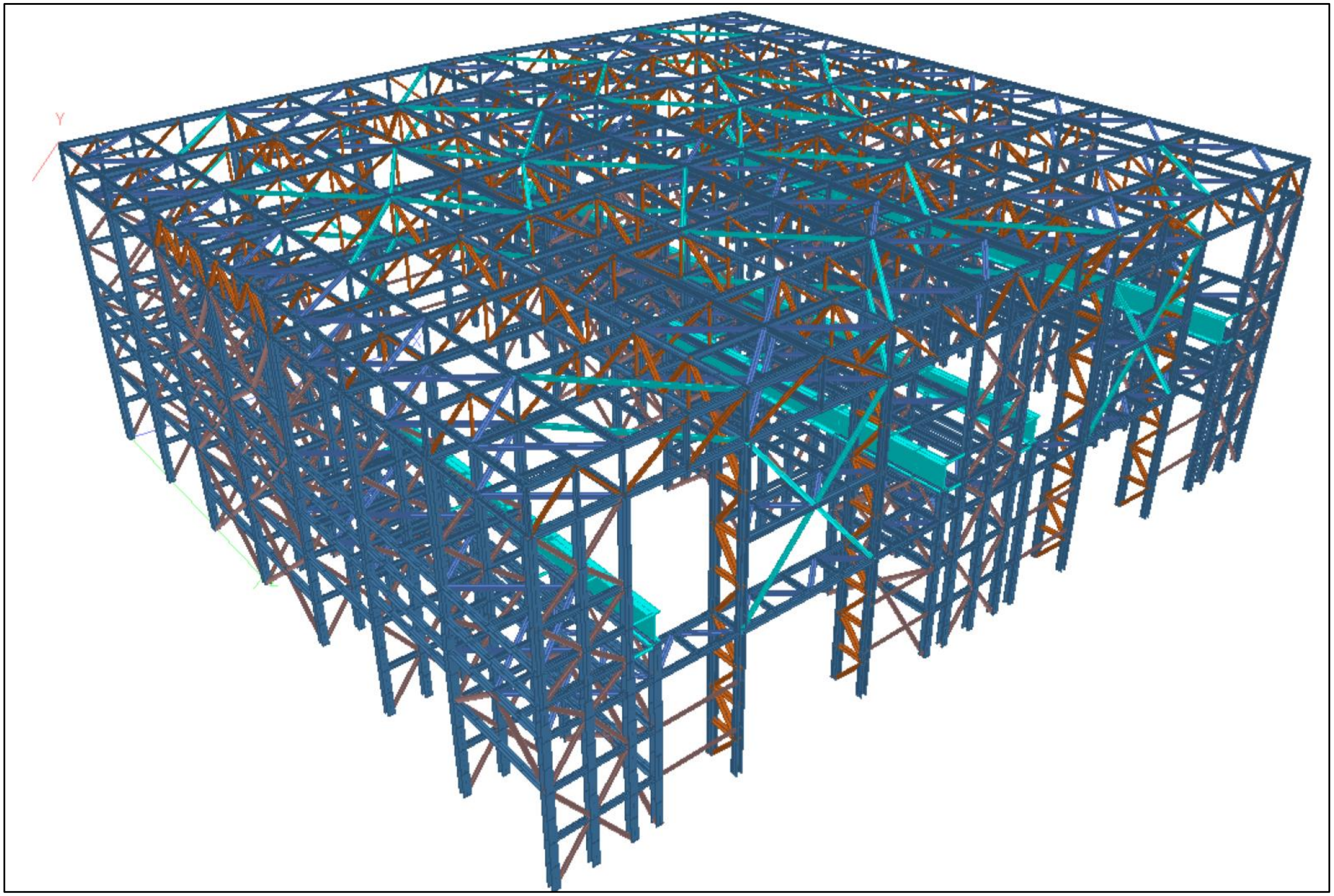

Figure 7-54
Isometric View of Cask Handling Building Structural Steel Framing

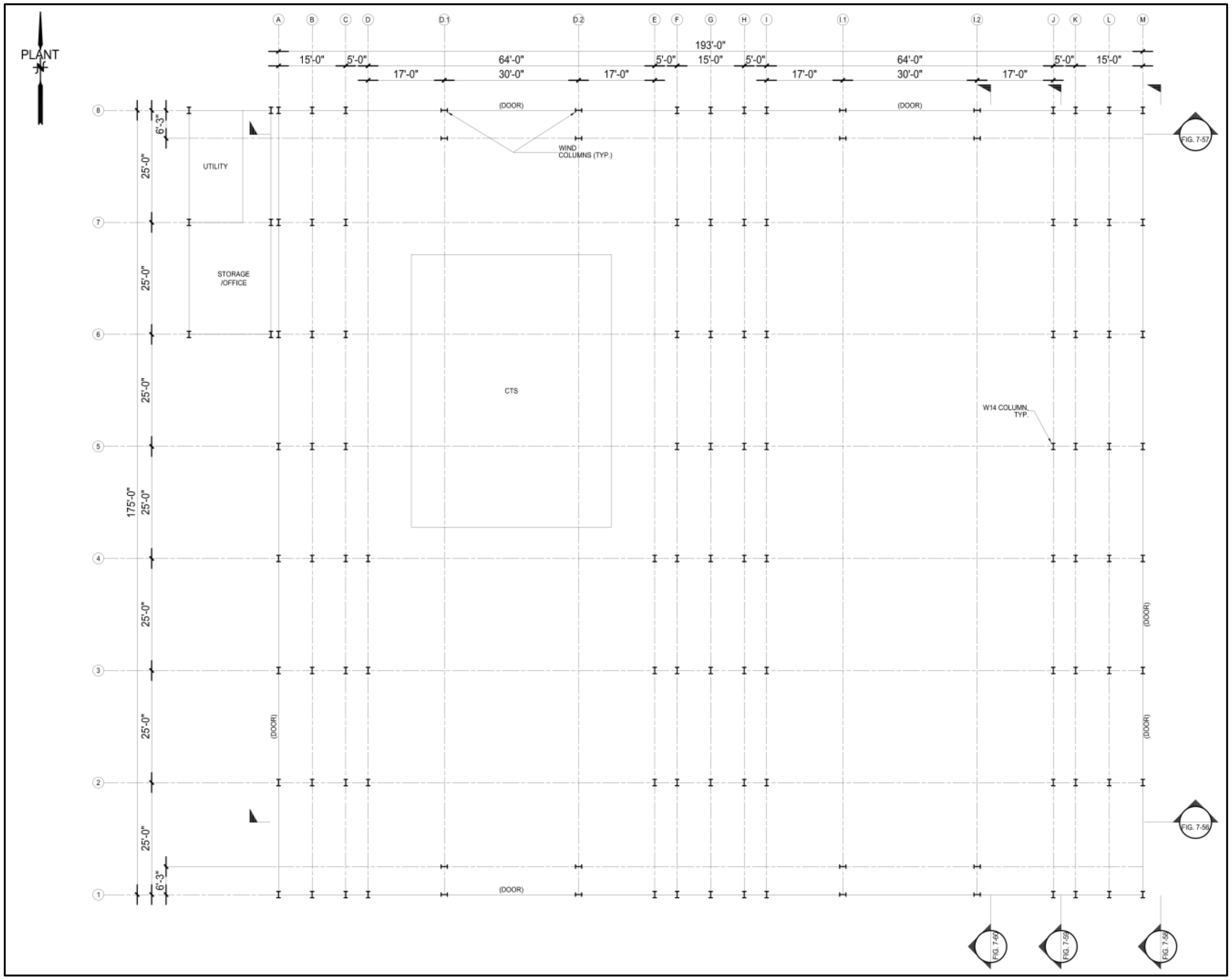

Figure 7-55
Plan View of Cask Handling Building Structural Steel Framing Arrangement, at Grade Level (Elevation 100'-0")

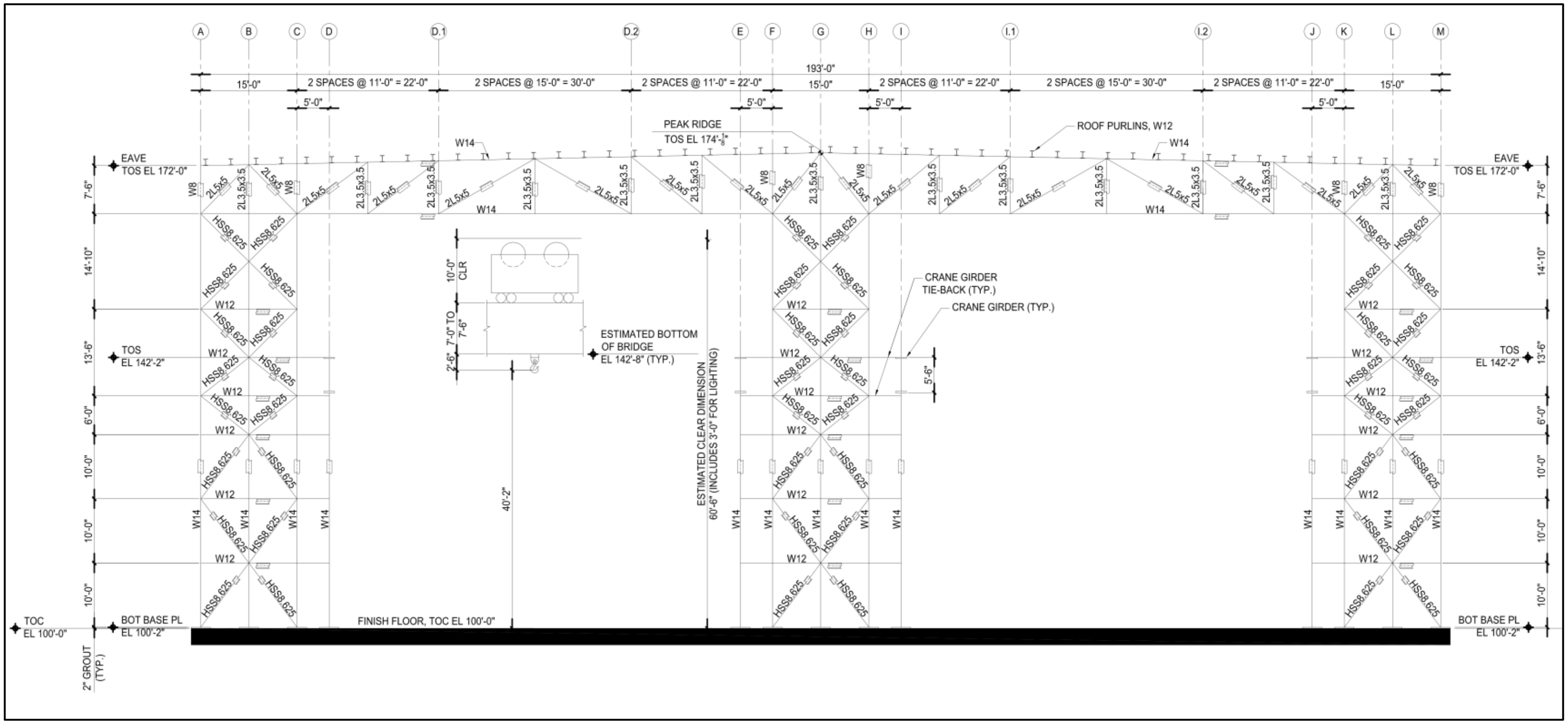

Figure 7-56
Cask Handling Building Structural Steel Framing Arrangement, Typical Interior Section (Looking North)

All Indicated Changes are in response to RAI NP-7-12

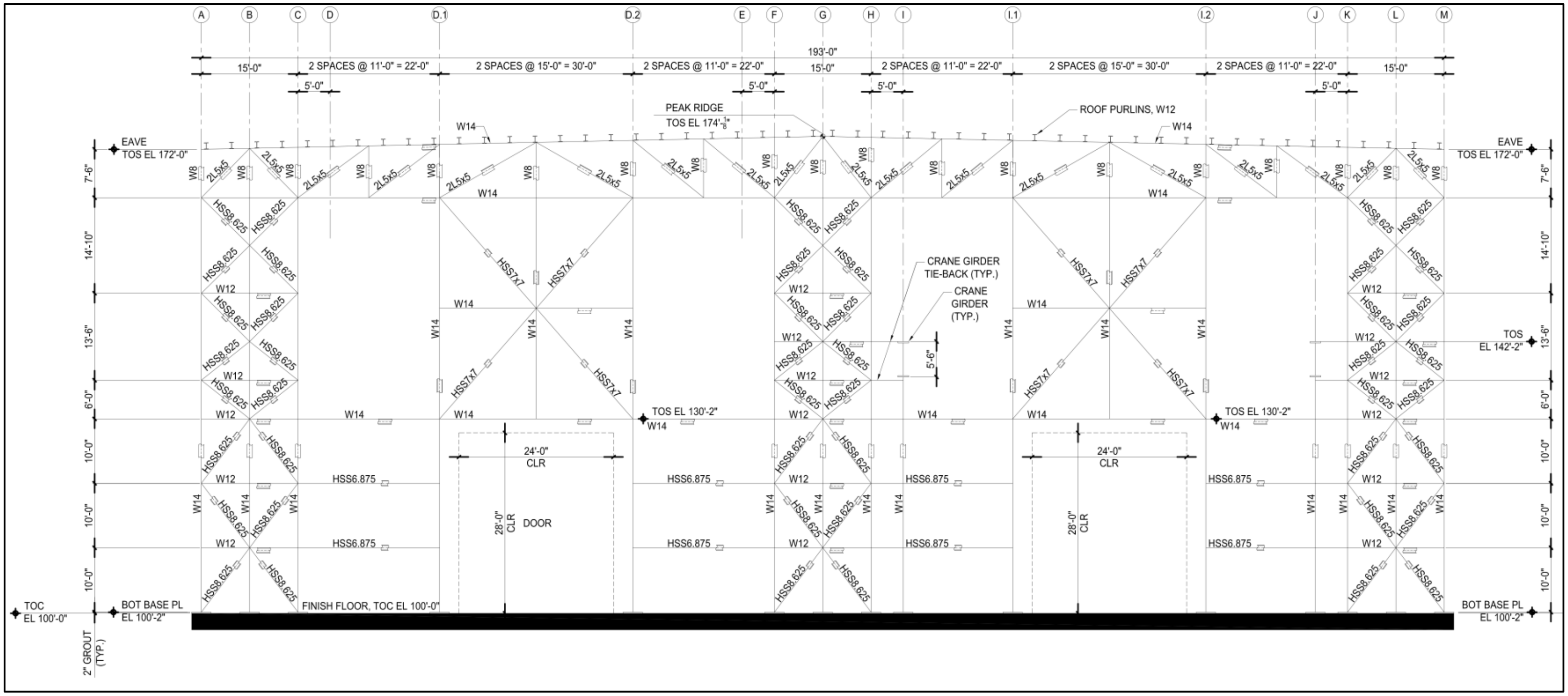

Figure 7-57

Cask Handling Building Structural Steel Framing Arrangement, Section at North Exterior Frame (Looking North)

All Indicated Changes are in response to RAI NP-7-12

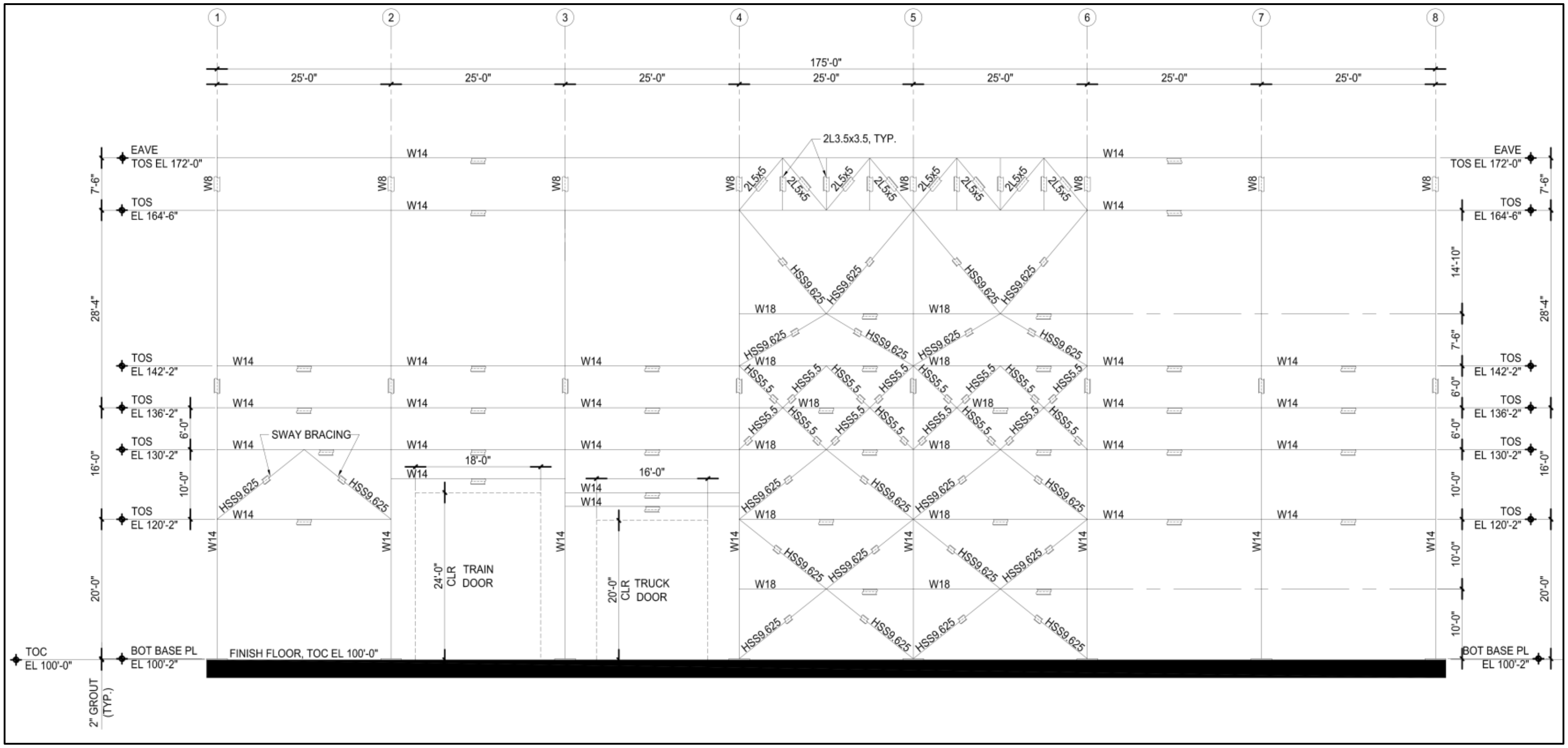

Figure 7-58
Cask Handling Building Structural Steel Framing Arrangement, Typical Section at Main Building Column Line (Looking West)

All Indicated Changes are in response to RAI NP-7-12

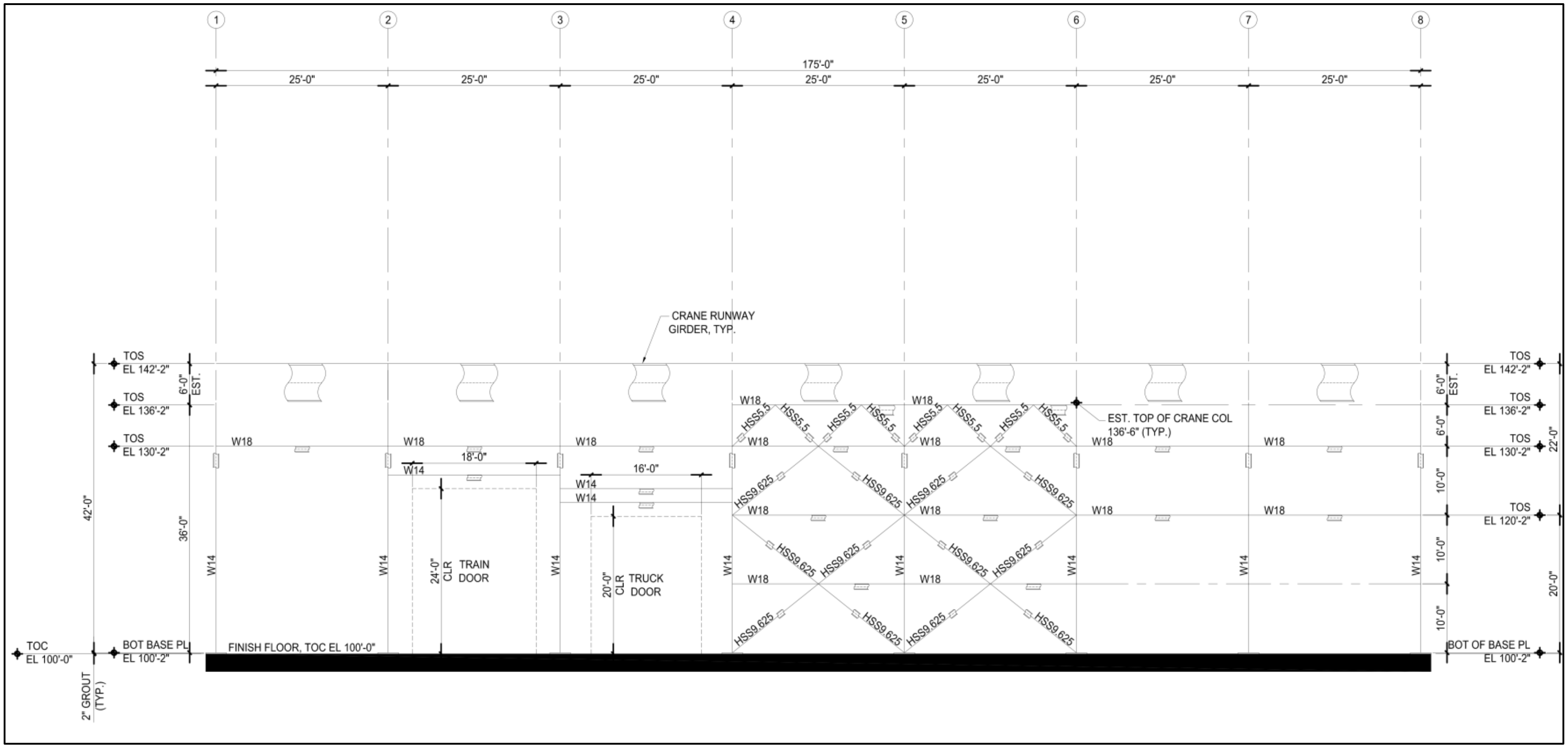

Figure 7-59
Cask Handling Building Structural Steel Framing Arrangement, Typical Section at Crane Column Line (Looking West)

All Indicated Changes are in response to RAI NP-7-12

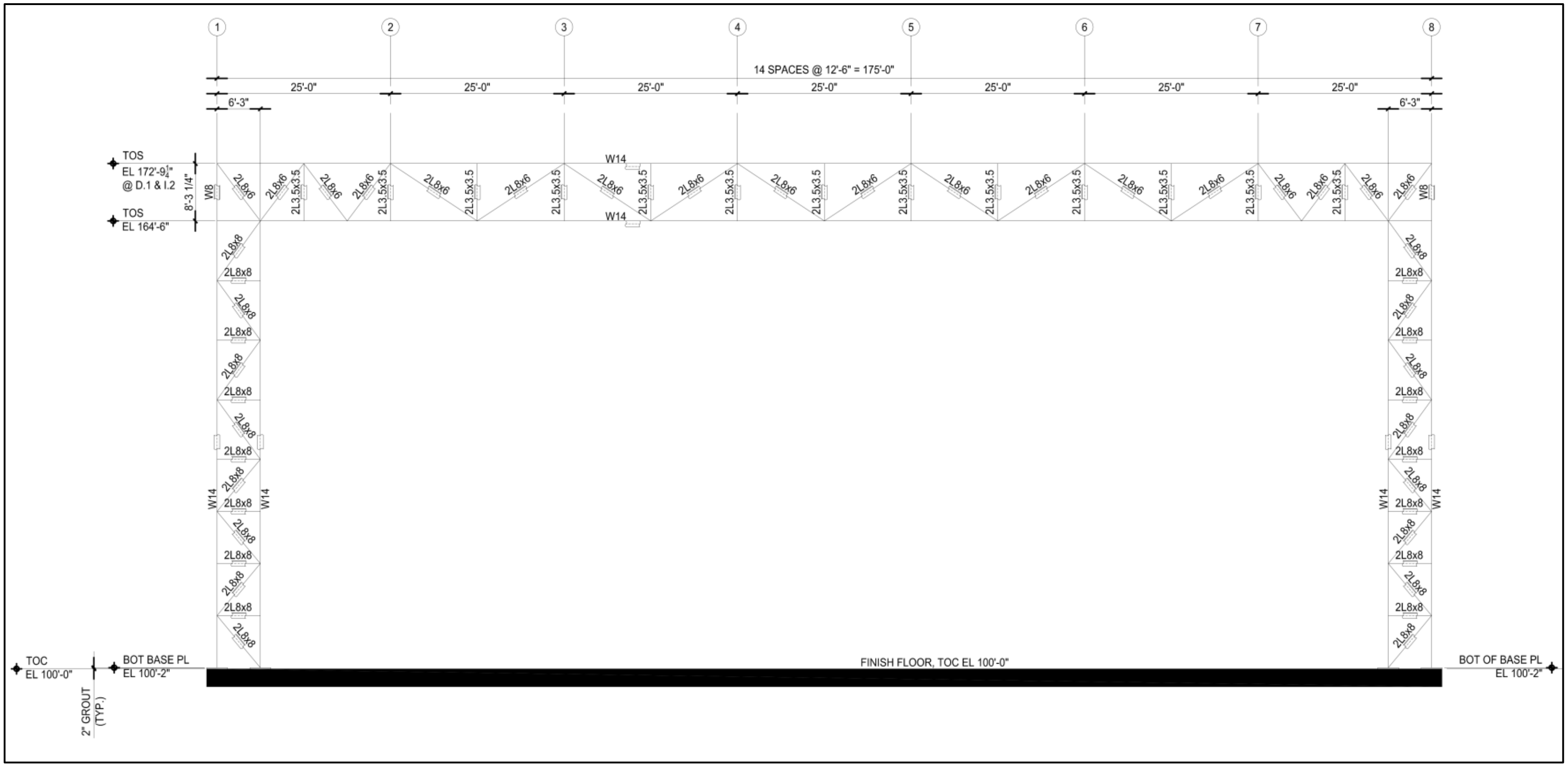

Figure 7-60
Cask Handling Building Structural Steel Framing Arrangement, Typical Section at Wind Column Line (Looking West)

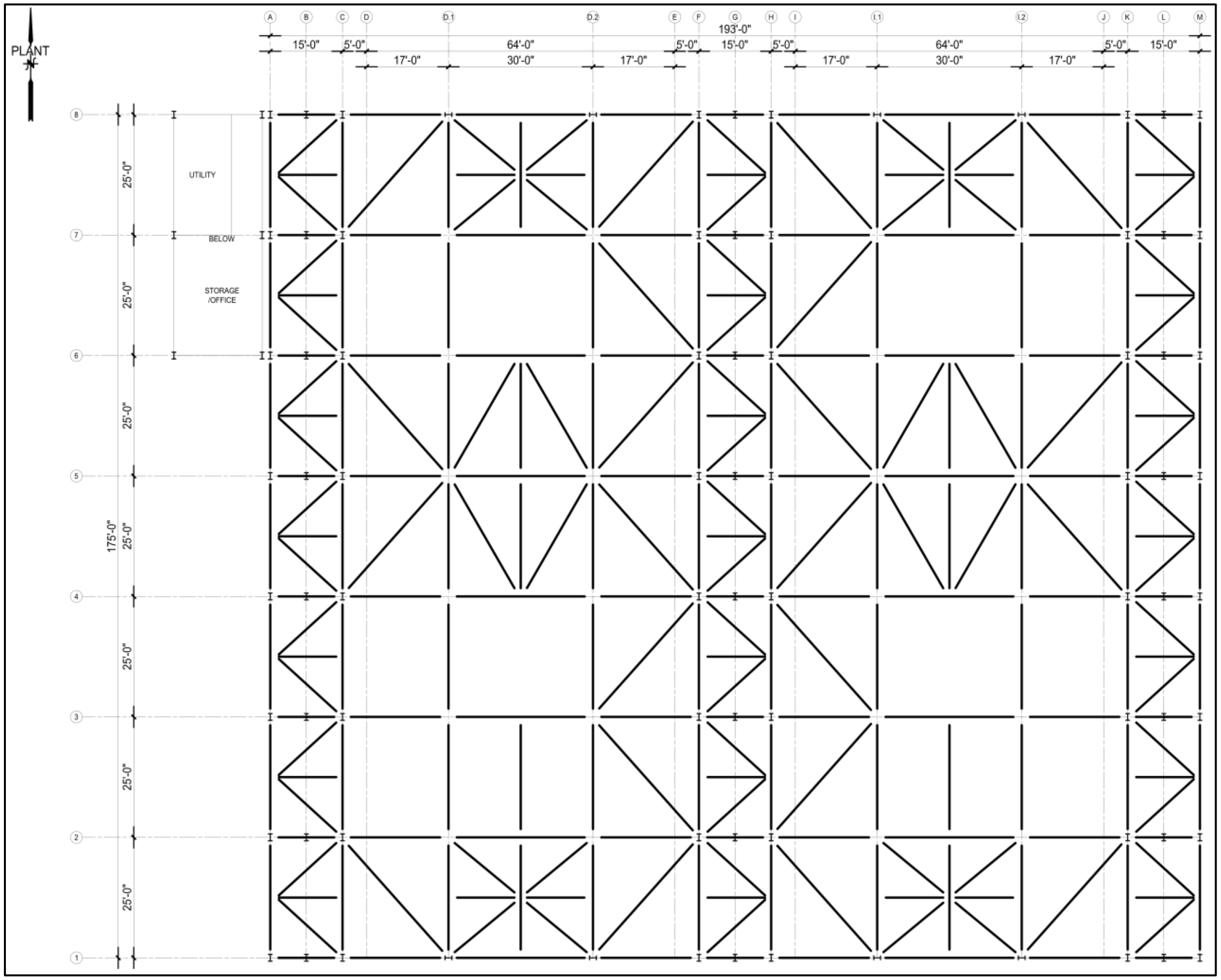

Figure 7-61
Cask Handling Building Structural Steel Framing Arrangement, Plan View at Roof Top Chord (Bottom Chord Similar)

All Indicated Changes are in response to RAI NP-7-12

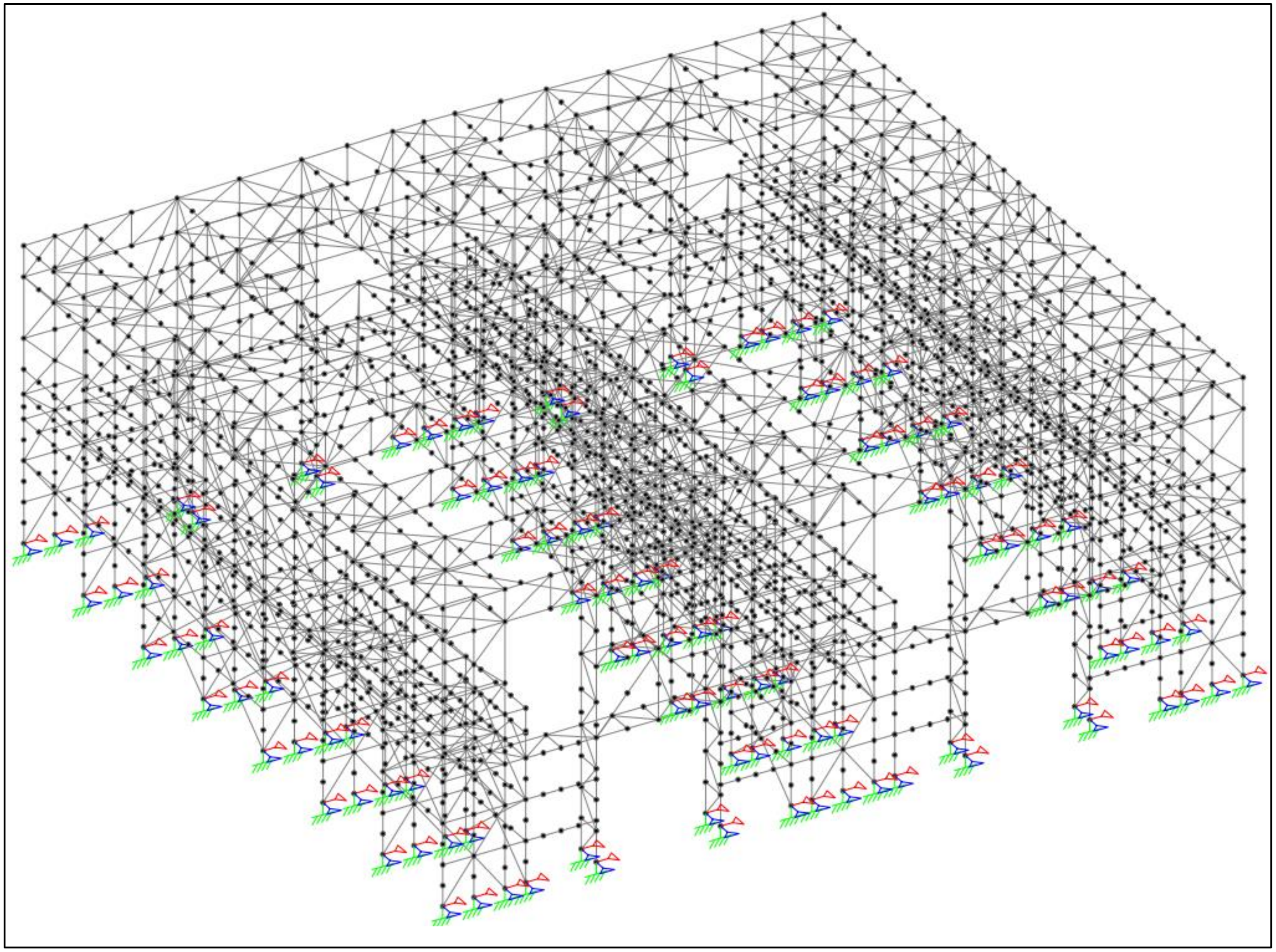

Figure 7-62
Cask Handling Building 3D STAAD.Pro Finite Element Model

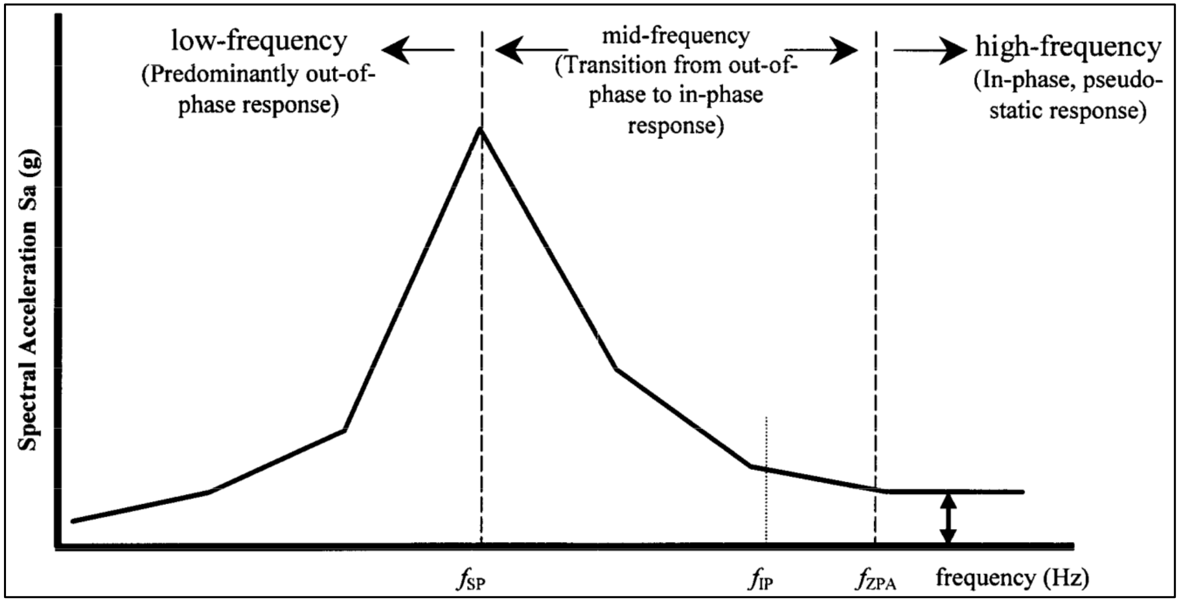

Figure 7-63
Generalized Acceleration Response Spectrum

Because only previously loaded canisters will be accepted at the WCS CISF the following topics identified in ISG-15 are remain unchanged from what has been previously reviewed and approved by the US NRC in the applications incorporated by reference listed in Section 1.6.

- Material Properties
- Weld Design and Inspection
- Galvanic and Corrosive Reactions
- Bolt Applications
- Protective Coatings and Surface Treatments
- Neutron Shielding Materials
- Materials for Criticality Control
- Seals
- Low Temperature Ductility of Ferritic Steels
- Fuel Cladding, including burnup and cladding temperature limits
- Prevention of Oxidation Damage During Loading of Fuel
- Flammable Gas Generation
- Canister Closure Weld testing and Inspection

15.1.5 Cask Handling Building

The materials used in the construction of the Cask Handling Building are given in Table 15-1.

15.2.2.2 AHSM

The reinforced concrete AHSM is designed to meet the requirements of ACI 349-97. Load combinations specified in ANSI 57.9-1984, Section 6.17.3.1 are used for combining normal operating, off-normal, and accident loads for the AHSM.

15.2.2.3 HSM Model 102

The HSM Model 102 reinforced concrete is designed to meet the requirements of ACI 349-85 and ACI 349-97 Editions, respectively. Load combinations specified in ANSI 57.9-1984, Section 6.17.3.1 are used for combining normal operating, off-normal, and accident loads for the HSM.

15.2.2.4 NAC-MPC VCC

The American Concrete Institute Specifications ACI 349 (1985) and ACI 318 (1995) govern the NAC-MPC system VCC design and construction, respectively.

15.2.2.5 NAC-UMS VCC

The American Concrete Institute Specifications ACI 349 (1985) and ACI 318 (1995) govern the NAC-UMS system VCC design and construction, respectively.

15.2.2.6 MAGNASTOR VCC

The American Concrete Institute Specifications ACI-349 (1985) and ACI-318 (1995) govern the MAGNASTOR system VCC design and construction, respectively.

15.2.3 Transfer Casks for Vertical Systems

The ANSI N14.6 (1993) and NUREG-0612 govern the NAC-MPC, NAC-UMS and MAGNASTOR system transfer cask designs, operations, fabrication, testing, inspection, and maintenance.

15.2.4 Cask Handling Building

Materials for Cask Handling Building steel structures will be constructed to ANSI/AISC 360-16. Materials for the Cask Building Overhead Cranes will adhere to NOG-1-2015 fracture toughness requirements. The reinforced concrete structures in the Cask Handling Building are designed to ACI 349-13 and constructed to ACI 318-08.

All Indicated Changes are in response to RAI NP-7-12

Coefficient of Thermal Expansion, α (x 10^{-6} in/in/°F)	5.9
Density (lbm/in^3)	0.29

15.3.4 Cask Handling Building

The Cask Handling Building is built with the use of reinforced concrete for foundation and slab, and structural steel members for above-ground structure.

The specifications and details that apply to these materials are given in Table 15-2.

All Indicated Changes are in response to RAI NP-7-12

Table 15-1
Material Specifications for Cask Handling Building Structures

Structural Element	***Applicable Material Specification***
Wide Flange Beams and Columns	*ASTM A992 Grade 50*
Channels	*ASTM A572 Grade 50*
Angles	*ASTM A572 Grade 50*
Plate	*ASTM A572 Grade 50*
Hollow Structural Shapes	*ASTM A1085*
Bolts for primary framing connections	*ASTM F3125 Grade A325*
Crane Rail	*ASTM A759*
Anchor Rods	*ASTM A193 Grade B7*
Concrete Reinforcing Steel	*ASTM A706 Grade 60*

Table 15-2
Material Properties for Cask Handling Building Structural Analysis and Design

Structural Element	***Property***	***Value***
Structural Steel Members and Plates	*Elastic Modulus, E*	*29,000 ksi*
	Poisson's Ratio, μ	*0.30*
	Coefficient of Thermal Expansion, α	*6.5 x 10-6 in/(in°F)*
	Unit Weight, γ	*0.490 kip/ft^3*
	Specified Yield Strength, F_y	*50 ksi*
Concrete Foundation and Slab	*Specified Compressive Strength, f'_c*	*4500 psi*
	Elastic Modulus, E	*3820 ksi*
	Poisson's Ratio, μ	*0.17*
	Coefficient of Thermal Expansion, α	*5.5 x 10-6 in/(in°F)*
	Unit Weight, γ	*0.150 kip/ft^3*
Concrete Reinforcing Steel	*Specified Yield Strength, F_y*	*60 ksi*
Anchor Rods	*Specified Yield Strength, F_y*	*105 ksi*
	Specified Tensile Strength, F_u	*125 ksi*
Structural Fill	*Unit Weight, γ*	*0.110 kip/ft^3*

All Indicated Changes are in response to RAI NP-7-12