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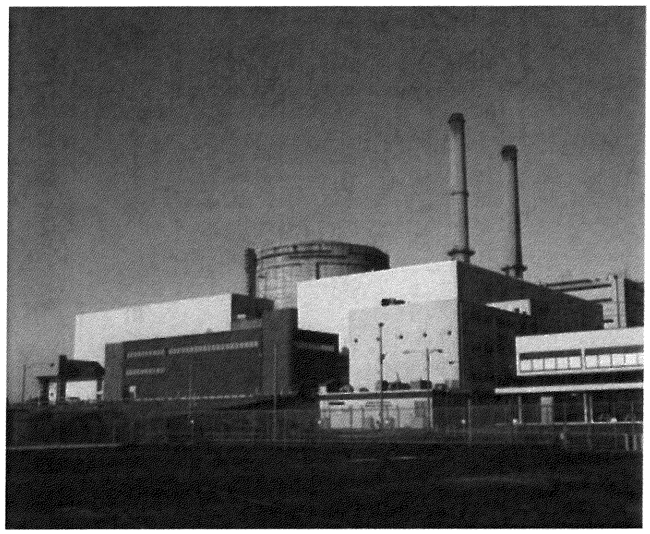
DESIGN CRITERIA DOCUMENT

For

CRYSTAL RIVER UNIT 3

AUXILIARY BUILDING EVALUATION FOR CRANE UPGRADE



Independent Review Required YES NO

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PROJECT REPORT REVISION STATUS

REVISION	**DATE**	**DESCRIPTION**
0	10/28/2010	Initial Issue
1	3/17/2011	Revised in its entirety
2	08/08/2011	Updated Impact Loads to be applied simultaneously in all three directions

PAGE REVISION STATUS

PAGE NO.	**REVISION**	**PAGE NO.**	**REVISION**
All	2		

APPENDIX REVISION STATUS

APPENDIX NO.	**PAGE NO.**	**REVISION NO.**	**APPENDIX NO.**	**PAGE NO.**	**REVISION NO.**
1	2	1			
2	14	2			
3	3	2			
4	3	1			

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1. INTRODUCTION

The Auxiliary Building at Progress Energy Crystal River Unit 3 (CR3) has a Whiting Overhead Crane (FHCR-5) with one non-single failure proof main hook and one non-single failure proof auxiliary hook. The main hook was originally rated for 120 tons (Ref. 14.5.1) but has subsequently been derated by 40% to 72 tons and then further derated to 25 tons (Ref. 14.1.1). To support future Dry Fuel Storage campaigns, it is required that the crane be upgraded to 130-tons and single failure proof status to support the loading and transferring of spent fuel into the TransNuclear supplied equipment. To achieve the necessary upgrades, a new crane, including the crane bridge structure and trolley will be provided to replace the existing crane in support of the Independent Spent Fuel Storage Installation (ISFSI).

The building consists of steel braced frames on a concrete support structure that forms the lower portion of the building, as shown in Figure 1. The footprint of the steel building is 208’-9” (N-S) by 48’ (E-W). The steel support structure consists of several floors and areas that vary from EL 119’-0” to 209’-1”. The column bases of the steel frame interface with the concrete support structure at EL 119’-0”, EL 143’-0”, and EL 162’-0”. The steel columns are W36 members that step to W14 members at EL 190’-0 1/4” and continue up to EL 209’-1” to support the steel roof structure. The crane runway girders are supported at the fabricated step in the building columns. The crane girders crane rails have a top elevation of 193’-7”.

The Auxiliary Building, with the exception of the steel roof support structure, is designated as a Class I structure (Ref. 14.1.2, Section 5.1). The concrete portion of the Auxiliary Building has been designed for the loads listed in the FSAR (Ref. 14.1.2, Section 5.4.1.2), which include Maximum Hypothetical Earthquake (MHE) and tornado loads. The steel support structure of the Auxiliary Building (from the 143’ to the 209’ elevation) including the building siding and roof, is not a Class I structure. As such, it is not designed or licensed to withstand tornado loads or to Class I seismic requirements. As the Auxiliary Building’s steel structure is not classified as a Class I or II structure, it is by default Class III in accordance with the FSAR (Ref. 14.1.2, Section 5.1.1.3). Based on a review of the original design calculations, the steel support structure was designed to withstand Operational Basis Earthquake (OBE) loads based on Ground Response Spectra. However, it was not designed to withstand Safe Shutdown Earthquake (MHE) loads.

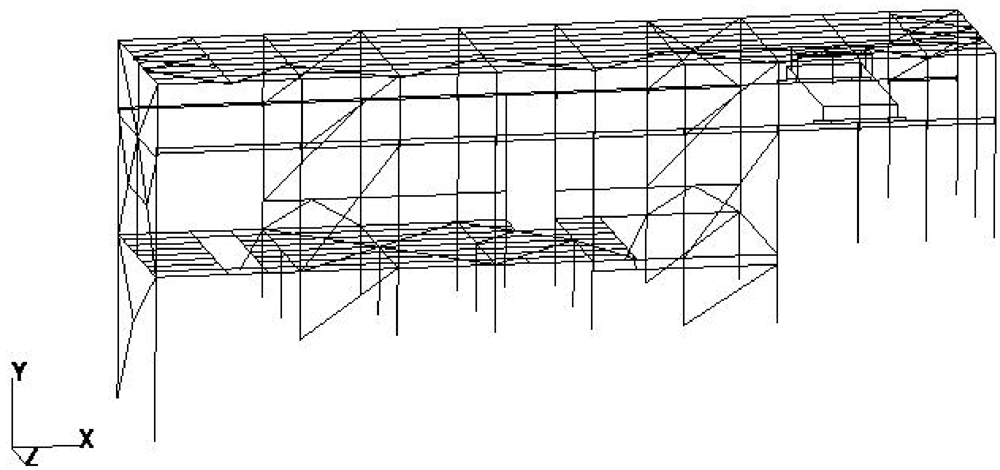


Figure 1: Overall geometry of the Auxiliary Building including the stick model of the crane.

2. SCOPE

The purpose of this Design Criteria Document (DCD) is to specify the loads, load combinations, acceptance criteria, and analysis methodology for the evaluation of the Auxiliary Building superstructure including the crane runway and its steel support structure with the upgraded single failure proof Overhead Crane (FHCR-5).

The scope of this DCD includes the steel structural portions of the Auxiliary Building, located above the lower concrete portion of the building. As indicated in Section 1, all structural elements are included with the exclusions:

- Concrete floors and decking at El. 162’-0” (Mass effects to be included)
- Building roof decking and roofing (Mass effects to be included)
- Girts and siding (Mass effects to be included)

The DCD is not applicable to the concrete portion of the building, which is not being reevaluated. Additionally this DCD is not applicable for the design of the crane. However, since a building/crane coupled analysis is required, this DCD does provide acceptance criteria for compatibility of the GT STRUDL model (to be used for building analysis) with the ANSYS model to be used by crane vendor for design of the crane.

3. ANALYSIS METHODOLOGY

The upgrade of the Overhead Crane (FHCR-5) in the Auxiliary Building at Crystal River Unit 3 requires a dynamic analysis of the steel frame that supports the new upgraded 130-ton crane. The evaluation and analysis of the Auxiliary Building steel structure will require a new calculation that will supplement the existing Auxiliary Building Gilbert Calculations. The analysis is required to establish that the Auxiliary Building steel structure is qualified for the crane in accordance with the current plant licensing basis and applicable provisions of NUREG-0554 (Ref. 14.2.5) as stipulated by ASME NOG-1 (Ref. 14.2.9).

An analysis model for the Auxiliary Building steel structure will be developed using GT STRUDL (Ref. 14.4.1) that includes a stick model of the crane modeled in accordance with ASME NOG-1 (Ref. 14.2.9) with properties provided by the crane vendor. The crane model will be included in the analysis since the mass of the crane is large with respect to the Auxiliary Building steel structure and the decoupling criteria specified in ASME NOG-1 (Ref. 14.2.9) cannot be met. Concurrently, an ANSYS model of the building will be prepared that will be compatible with the GT STRUDL model. The ANSYS model will be utilized by the crane vendor for design of the crane and associated components.

The following analysis methodology is developed and summarized in Appendix 1. The Auxiliary Building steel structure shall be completely modeled and evaluated using GT STRUDL and shall include the stick model of the crane. Placement of the crane bridge and trolley on steel supporting structure is selected in such a way that it captures the critical responses for design of the Auxiliary Building steel structure. See Section 10 for more details. This GT STRUDL analysis shall be used to identify and incorporate any building modifications that may be necessary and to qualify the Auxiliary Building for the upgraded crane.

The ANSYS model will include any proposed modifications to the building consistent with the GT STRUDL analysis. Documentation of the ANSYS model and associated inputs, computer runs for compatibility verifications, etc. will be documented as an independent calculation, separate from the building evaluation.

4. CURRENT DESIGN BASIS

The Auxiliary Building will be analyzed in accordance with the existing calculations of record, the FSAR (Ref. 14.1.2), and AISC (Ref. 14.2.2). The 2:01 calculations are applicable to the Auxiliary Building, including the following (Refs. 14.3.1 to 14.3.11):

2:01.7D	Applied Load from Steel Structure
2:01.10	Steel Frames
2:01.11	Steel Columns
2:01.12	Vertical Bracing
2:01.13	Crane Runway Beams
2:01.14	Steel Floor Framing @ EL. 162’-0”
2:01.15	Roof Framing, Girts, and Miscellaneous Steel
2:01.16	Seismic Analysis of Steel Frame
2:01.48	Basic Design Requirements – Aux Bldg
2:01.50	Structural Steel – Aux Bldg
2:01.55	Support Walls and Columns – Aux Bldg

No degradation of the steel and concrete structures will be considered in the building analysis. It is expected that the building structure is maintained in satisfactory condition consistent with plant maintenance requirements and existing site procedures.

5. APPLICABLE CODES AND STANDARDS

The evaluation of the existing Auxiliary Building steel structure, and the design of new structural steel, including modifications to existing steel, shall conform to the original plant licensing basis documents (Refs. 14.1.2, 14.3.1 to 14.3.11) including all the requirements of the AISC Code (Ref. 14.2.2).

The 6th edition of the AISC Code (Ref. 14.2.2) does not provide any specific methodology to account for the prying action of beam and column connections. Therefore, an evaluation of prying action for connection design shall be based on the methodology provided by the 9th edition of the AISC Code (Ref. 14.2.12).

Type A490 Bolts may be used for the building modifications. A490 bolts are not provided in the 6th edition of the AISC Code (Ref. 14.2.2). Therefore, the bolt allowables from the 9th edition of the AISC Code (Ref. 14.2.12) shall be used.

Material for modifications to existing structural steel shall conform to ASTM Specification A36 in accordance with drawings and specifications (Refs. 14.1.5, 14.1.6 and 14.5.70).

6. MATERIAL PROPERTIES

The material properties used for the analysis of the Auxiliary Building are shown below in Table 1.

Table 1: Material properties of existing structural elements in the Auxiliary Building

Material	Properties	Reference
Structural Steel ASTM A36	F_y = 36,000 psi	SP-5757, RO-2968, 522-001 (Refs. 14.1.5, 14.1.6 & 14.5.70)
	E = modulus of elasticity = 29,000,000 psi	AISC (Ref. 14.2.2)
	Poisson's Ratio = 0.3	AISC (Ref. 14.2.2)
	Mass Density = 490 lb/ft^3	AISC (Ref. 14.2.2)
Structural Weld E70XX	F_u = 70,000 psi	SP-5757, RO-2968, 522-001 (Refs. 14.1.5, 14.1.6 & 14.5.70)
Anchor Bolts A36	F_y = 36,000 psi	AISC (Ref. 14.2.2)
Anchor Bolts A449	F_y = 58,000 psi	Calc. 2:01.10 (Ref. 14.3.2)

7. LOADS

7.1. Dead Loads

Dead loads will consist of the self-weight of structural members including the supporting steel and concrete, girts and siding, purlins, roofing, and miscellaneous equipment.

The dead load of the crane (e.g., trolley, bridge girders, and additional attachments) will be provided by the crane vendor and included in the model as described in Section 11.

7.2. Floor Live Loads

At elevation 162’-0”, a 300 psf live load is considered in accordance with DBD 1/3 (Ref. 14.1.3).

7.3. Roof Live Loads

An area roof live load at EL 209’-1” of 30 psf is used as specified in DBD 1/3 (Ref. 14.1.3).

7.4. Crane Live Loads

The crane live load will consist of a maximum of 130 tons for the main hook and 15 tons for the auxiliary hook (Ref. 14.4.3). The loads of main hook and auxiliary hook are not concurrent. Therefore, only the main hook load is considered in the structural frame analysis.

7.5. Crane Impact Loads

Impact loads resulting from the operation of the crane are applied to the structural model in accordance with DBD 1/3 (Ref. 14.1.3) and ASME NOG-1 (Ref. 14.2.9). Gilbert Calculation 2.01.13 (Ref. 14.3.5) uses the impact loads listed in DBD 1/3 for analysis and the impact loads are applied independently in each direction. For the load combinations listed in ASME NOG-1 (Ref. 14.2.9), the impact loads are applied simultaneously in all three directions.

See Table 2 below for the summary of Sections 7.5.1 to 7.5.3.

Table 2: Recommended impact factors

Crane Impact Loads	ASME NOG-1 (Ref. 14.2.9)	DBD 1/3 (Ref. 14.1.3)	Factors Used in Analysis**
Vertical Impact Load	15 (Percent of max lift load)	25 (Percent of max lift load)	DBD 1/3
Transverse Impact Load	10 (Percent of trolley and lift load – which is the longitudinal horizontal load on the crane bridge girders)	20 (Percent of trolley and lift load – 10% applied to each crane runway girder)	DBD 1/3
Longitudinal Impact Load	5 (Percent of gantry bridge, trolley load and lifted load – which is the transverse horizontal load on the crane bridge girders)	10 (Percent of max wheel load)	DBD 1/3

** see Section 7.5.1 to 7.5.3 for explanation.

7.5.1. Vertical Impact Load

DBD 1/3 (Ref. 14.1.3) defines twenty-five percent of lift loads as Vertical Impact and ASME NOG-1 (Ref. 14.2.9) defines fifteen percent of lift load as Vertical Impact. As the factors defined in DBD 1/3 envelopes the factors defined in ASME NOG-1, they shall be used in the analysis.

7.5.2. Transverse Impact Load

The transverse direction is defined as the direction perpendicular to crane runway girder and which generates horizontal loads on the crane runway girder.

7.5.3. Longitudinal Impact Load

The longitudinal direction is defined as the direction along the crane runway girder.

7.6. Seismic Loads

Seismic loading shall include self excitation of the mass of the building and crane structures, including the rated lift load. Additionally, ten percent (10%) of the floor live load at floor elevation 162'-0" in the building model shall be considered as excitable mass in the dynamic analyses.

7.6.1. Seismic Response Spectra

An operating basis earthquake (OBE) peak ground acceleration of 0.05g horizontal and 0.033g vertical will be used consistent with the Crystal River Unit 3 FSAR (Ref. 14.1.2).

A maximum hypothetical earthquake (MHE) peak ground acceleration of 0.10g horizontal and 0.067g vertical will be used consistent with the Crystal River Unit 3 FSAR (Ref. 14.1.2).

The original design of the Auxiliary Building steel structure per Gilbert Calculation 2:01.10 (Ref. 14.3.2) and Calculation 2:01.16 (Ref. 14.3.8) uses ground level OBE response spectra, applied at the anchorage to the concrete portion of the building. Seismic coefficients were used to develop equivalent static seismic forces at the various floor elevations. These forces were then used to design the various structural members of the Auxiliary Building. Although a damping value of 2.5 percent is specified in the FSAR (Ref. 14.1.2) for bolted steel structures, which would apply to the Auxiliary Building steel structure, 1% damping value was used in the original building design.

In order to ensure that the building qualification is compatible with the requirements for the design of the crane structure, additional seismic requirements have been imposed. Specifically, ASME NOG-1 (Ref. 14.2.9) requires that the seismic input be a broadened floor response spectra defined at an appropriate level in the structure supporting the crane. Since a coupled building/crane analysis is required, the response spectra would correspond to the anchorage locations of the Auxiliary Building steel structure. ASME NOG-1 specifies the damping values to be used in the crane design as 7 percent of critical damping for MHE (SSE) and 4 percent of critical damping for OBE.

In order to consider the bounding seismic inputs, the enveloping seismic inputs per the current design and those specified for the crane shall be utilized. Specifically, this would require enveloping the 1 (and 2.5) percent ground response spectra with the 4 percent OBE and 7 percent MHE (SSE) response spectra, respectively, for the OBE and MHE (SSE) conditions. These floor response spectra are enveloped in Appendix 2 of this document.

As discussed in Section 13, an ANSYS model will be generated for use by the crane vendor for detailed design of the crane. In order to provide reasonable latitude to address analytical differences between the GT STRUDL and ANSYS analysis results during comparison of seismic responses of the two models, the input acceleration values of the response spectra for the GT STRUDL analysis will be increased by 5 percent.

The floor response spectra for OBE with 4% damping and MHE with 7% damping are not available. The response spectra curves corresponding to these damping values have been obtained utilizing available floor response spectra and documented in Appendix 2.

7.6.2. Modal Combination

The combination of the modal responses of the Auxiliary Building steel structure will be in accordance with the CQC methodology as described in Regulatory Guide 1.92 (Ref. 14.2.8).

7.6.3. Zero Period Acceleration

Effect of ZPA shall be included in the building qualification, to account for modes higher than the ZPA frequency of 33 Hz. Zero period acceleration shall be applied to the missing mass in accordance with Regulatory Guide 1.92 (Ref. 14.2.8), and the results combined with the dynamic analysis using Square Root of the Sum of Squares (SRSS) method.

7.6.4. Directional Combination

The current licensing basis of the plant requires that the combination of seismic direction responses be the envelope of the absolute sum of the responses in the vertical and one horizontal direction (north-south or east-west) in accordance with the FSAR (Ref. 14.1.2). ASME NOG-1 (Ref. 14.2.9) requirements for crane design specify that the directional responses in the three orthogonal directions be combined using SRSS combination method. Since a coupled analysis of the building and crane is to be performed, as conservative/bounding approach, enveloping results from the following directional combinations shall be utilized:

- absolute sum of the responses in the vertical and one horizontal direction (north-south or east-west)
- SRSS combination of the responses in the three directions

7.7. Wind Loads

The wind loads shall be based on a design wind of 110 mph as established in FSAR (Ref. 14.1.2) and consistent with Gilbert Calculations 2:01.10 and 2:01.48 (Refs. 14.3.2 and 14.3.9). Per Section 5.2.1.2.5 of FSAR (Ref. 14.1.2) a design wind of 110 mph (at 30 feet above grade) is the fastest mile of wind with a 100 year period of recurrence and is consistent with Section 4134 (b) of ASME NOG-1 (Ref. 14.2.9). The wind load shall be applied simultaneously, as applicable, to the windward walls, leeward walls, the side walls, and the roof as determined by the pressure coefficients specified in ASCE Paper No. 3269 (Ref. 14.2.1) and shown in Table 3.

Table 3: Wind Coefficients applied to the Auxiliary Building

Wind Coefficients		
	North – South	East – West
Windward	0.90	0.90
Leeward	0.56	0.35
Side Walls	0.80	0.80
Roof	0.72	0.52

The pressure coefficient for the windward wall is 0.9 and for the side walls a pressure coefficient of 0.8 shall be used. The pressure coefficients for the leeward wall and the roof shall be linearly interpolated from the values given in ASCE Paper No. 3269 (Ref. 14.2.1) based on the height-width ratio of the building. The leeward coefficient is 0.3 for the height-width ratio of 0.25 and is 0.5 for a ratio of unity and is 0.6 for the ratio of 2.5 or greater. The coefficient used for entire roof is 0.5 for a height-width ratio of 0.25 and is 0.8 for a ratio of 2.5 or greater.

7.8. Operating Wind Load

An operating wind load will be based on a basic wind speed of 50 mph. ASCE 7-05 (Ref. 14.2.10) and NUREG-0800 (Ref. 14.2.11) are used to calculate the wind pressure for operating wind. The operating wind load will be applied to the Auxiliary Building steel structure in accordance with ASCE 7-05 (Ref. 14.2.10) and combined with independent loads per ASME NOG-1 Section 4140 (Ref. 14.2.9).

7.9. Thermal Loads

The building structure is thermally constrained only at the column attachments to the concrete structure. The building structure experiences a temperature range of 55ºF to 95ºF. Thermal expansion, considering an ambient temperature of 70ºF will be small and the structural configuration provides adequate flexibility. Consequently thermal expansion loads on the structure will be negligible. Therefore, thermal loads will not be considered in the analysis of the Auxiliary Building steel structure.

7.10. Sloshing of Fuel Pool Water

The sloshing of the water has no impact on the crane support structure because the sloshing will occur below the steel support structure at the concrete pool walls.

7.11. Tornado Effects

Effects of tornado wind and tornado generated missiles will not be considered, consistent with the current design of the Auxiliary Building steel structure.

7.12. Pendulum Effect

The pendulum effect of the lifted load on structure during a seismic event, as required by NUREG-0554 Section 2.5 (Ref. 14.2.5), will be considered in the analysis of the Auxiliary Building steel structure. The lifted load in the hook-up and hook-down position will be modeled to allow for the dynamic effects of the swinging mass.

8. LOAD COMBINATIONS

The load combinations for the steel structure shall be in accordance with the original Auxiliary Building Calculations and Section 4140 of ASME NOG-1 (Ref. 14.2.9) as shown in Table 4. The load combinations used in the building analysis and presented in Table 4 envelope the original calculations and applicable load combinations per ASME NOG-1 as shown in Appendix 3. As discussed in Section 7.11, tornado effects are not considered in the load combinations. The structural analysis shall analyze the structure with different crane configurations and the applicable load cases shall be applied, as required.

In addition to the load combinations shown in Table 4, a load case considering the effects of dead, live, crane live, and wind loads ($D + L + L_c + W$) will be considered, consistent with the original Gilbert Calculations. This load case will be conservatively considered, however procedural requirements of the crane operation will be established to prohibit crane operation during weather conditions in which the design wind load would occur.

Table 4: Load Combinations used to structurally qualify Auxiliary Building steel structure.

Load Combination	Allowable Stress Increase
$D + L + L_c$	None
$D + L + L_c + I_V$	None
$D + L + L_c + I_T$	None
$D + L + L_c + I_L$	None
$D + L + W$	1.33
$D + L + L_c + E$	1.33
$D + L + L_c + E'$	Elastic Limit
$D + L + L_c + I_V + I_T + I_L + W_O$	1.33
$D + L + L_c + E + W_O$	1.33
$D + L + L_c + E' + W_O$	Elastic Limit
$D + L + E + W_O$	1.33
$D + L + E' + W_O$	Elastic Limit

$L = L_f + L_r$

D	=	Dead Load Including Crane Members
L_f	=	Floor Live Load
L_r	=	Roof Live Load
L_c	=	Crane Live Load
W	=	Design Wind Load
W_O	=	Operating Wind Load
E	=	Earthquake Load (OBE)
E'	=	Earthquake Load (MHE) (Note: This is same as SSE)
$I_{V,T,L}$	=	Crane Impact Loads (vertical, transverse, longitudinal)

9. ALLOWABLE STRESSES

The allowable stresses are specified in Table 4 above. The allowable stresses for steel members, bolts, rivets and welds may be increased by one third under the loading produced by wind or seismic, acting alone or in combination with the design dead and live loads. This is consistent with Section 1.5.6 of AISC (Ref. 14.2.2). Under the abnormal condition when forces are produced by the maximum hypothetical earthquake (E') loading, stresses may be increased to the elastic limit consistent with DBD 1/3 (Ref. 14.2.9).

10. BUILDING STRUCTURAL MODEL

The structural analysis program, GT STRUDL (Ref. 14.4.1), will be used to develop the 3D structural model of the Auxiliary Building steel structure with the new crane upgrade and will perform the required static and dynamic analyses as set forth in this document.

The model will encompass the Auxiliary Building steel structure from Column Lines I_1 to S_1 in the N-S direction and Column Lines 301 to 302A in the E-W direction. The building will be modeled from the column bases at various elevations to the top of the roof at EL 209'-1". The structural details of the Auxiliary Building, crane runway, and support framing are shown in the Auxiliary Building drawings (Refs. 14.5.2 to 14.5.77).

The steel frame which supports the FHCR-5 crane is analytically decoupled from adjacent auxiliary steel frame at column line 302-A. The adjacent frame is not physically decoupled from the Auxiliary Building, however it consists of a lateral bracing system of steel brace frames and concrete shear walls and is sufficient to carry its own lateral loads, as shown in drawing 522-003 (Ref. 14.5.72). Therefore, the crane supporting steel frame is not required to provide lateral stiffness to the adjacent frame and both structures are considered to be self-sustaining. The effects of the contributing mass of the decoupled structure will be included by considering the effective tributary masses of the adjacent decoupled spans.

The 8” thick concrete floor at 162’-0” is not a safety related concrete and not connected with the steel structure. Consequently, the diaphragm effect of concrete floor is not included in the model. Since the floor is supported by the steel structure, the weight and mass of the floor will be included in the analysis.

The weight and mass of structural features not included in the model, such as, girts, siding, etc. are considered in the model as concentrated and/or distributed weight/mass. A portion of the live load at floor elevation 162’-0” shall be considered as excitable mass to account for the mass contribution of expected live loads on the floor.

Any required modifications that are identified by the analysis of the Auxiliary Building shall be incorporated in full to account for the change in the dynamic characteristics of the building.

11. CRANE MODEL

11.1. Input Parameters

The configuration of the crane will be modeled in the analysis based on information provided by the crane vendor, including geometry, end conditions, mass distribution, etc. A simplified ANSYS model of the crane shall be provided by the crane vendor as an input for the building analysis.

The geometry of the crane and the boundary conditions at the wheel locations shall comply with ASME NOG-1 (Ref. 14.2.9) and as shown in Figure 2 below. The boundaries at the contact of wheels and rails are modeled per ASME NOG-1, Table 4154.3-1 (Ref. 14.2.9). The restraint conditions at the nodes are listed in Table 5. Note that the boundary conditions will apply horizontal transverse seismic loading to one crane rail only. The load will be applied to the crane girder with the longest span to produce the worst-case loading.

Table 5: Restraint conditions at the crane nodes for the sign convention defined in Figure 2.

	Translation			Rotation		
Node	X	Y	Z	θ_x	θ_y	θ_z
A	Fixed	Fixed	Fixed	All Free		
B	Fixed	Free	Fixed			
C	Free	Fixed	Fixed			
D	Free	Free	Fixed			
E	Fixed	Fixed	Fixed			
F	Fixed	Fixed	Fixed			
G	Free	Fixed	Fixed			
H	Free	Fixed	Fixed			

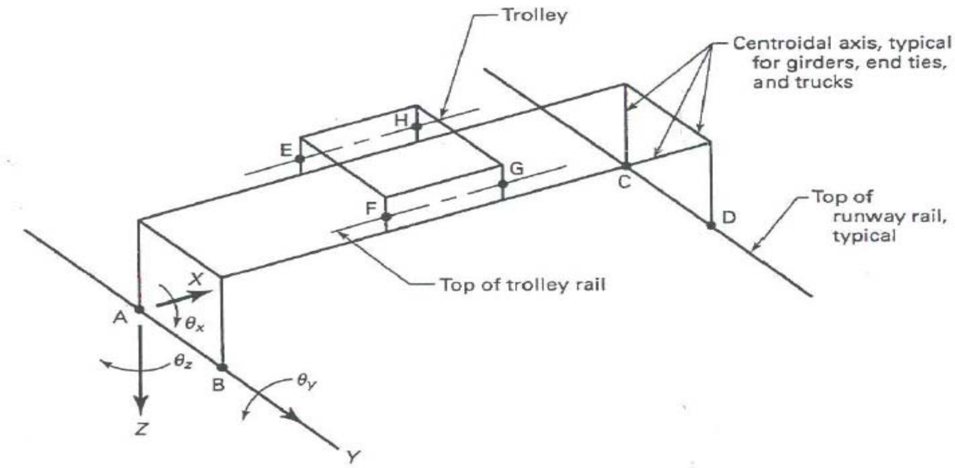


Figure 2: Crane Boundary Conditions, ASME NOG-1, Fig. 4154.3-1 (Ref. 14.2.9)

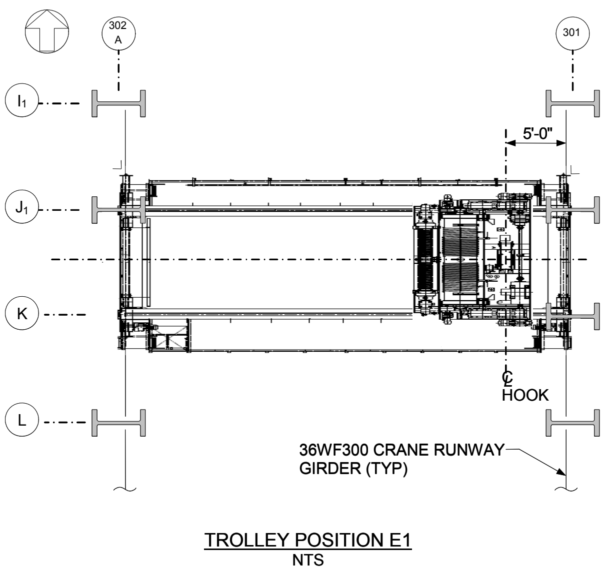
11.2. Trolley Locations

The analysis model will address various configurations of the crane bridge, trolley, and hook in order to obtain bounding responses of the structure. Per ASME NOG-1 (Ref. 14.2.9), the analyses are to be performed with the trolley at its extreme end positions on the bridge span, the trolley at the quarter points of the span positions, and trolley at mid span. However, since the quarter point and end position of the trolley on the west end of the bridge span are almost identical (10'-7 ¾" for the quarter point compared to 11'-6" for the end position), these positions can be combined into one configuration in a conservative manner. The quarter point positioning of the crane trolley, as specified by ASME NOG-1, is used to insure that all relevant peaks of the response spectrum are considered in the analysis. As the building is reasonably symmetrical about the north-south axis, the combination of the quarter point with the end point is valid. The four trolley configurations are shown in Figure 3.

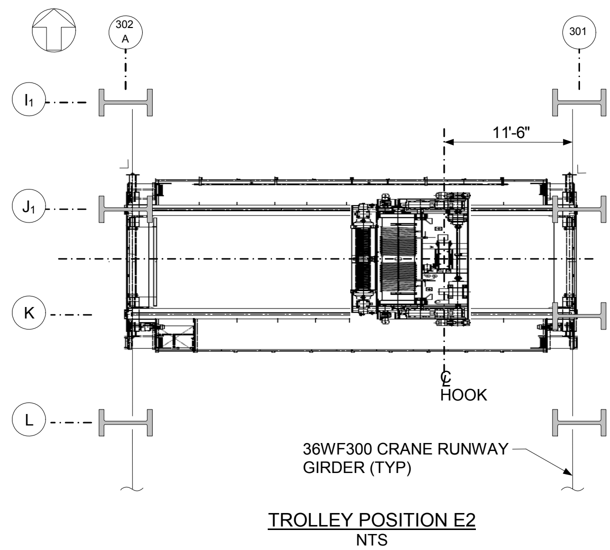
11.3. Bridge Locations

Various crane bridge positions are selected in order to maximize the structural responses of the Auxiliary Building due to moving crane loads as described in Table 6. Each crane bridge position will be combined with different trolley positions, and hook positions.

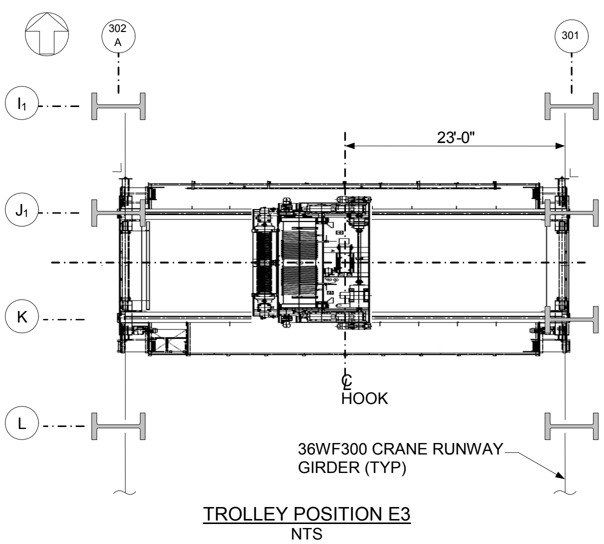
The vertical acceleration of the hook due to the maximum seismic loading will be assessed to determine if a slack rope condition exists. At each bridge location, the structure will initially be analyzed with the trolley at different locations (i.e., each end, mid-span and the quarter point from the east side). The calculation will account for the loaded and unloaded hook up and loaded hook down.



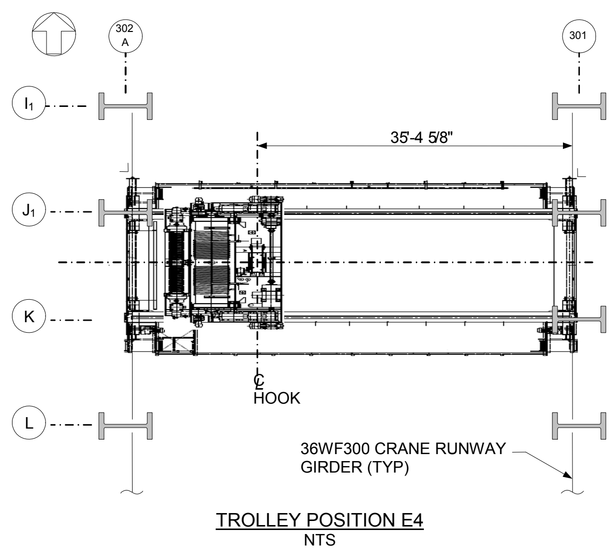
(a) Trolley Position E1



(b) Trolley Position E2



(c) Trolley Position E3



(d) Trolley Position E4

Figure 3: Modeled Trolley Positions

Table 6: Descriptions of the various crane bridge positions

Crane Bridge Position		Description
Long Span on the North End of the Auxiliary Building between column lines L and J_1	N1	Maximum Moment
	N2	Maximum Shear
	N3	Maximum Column Load
Typical Span between column lines Q_1 and L	N4	Maximum Shear
	N5	Maximum Column Load
	N6	Maximum Moment
Long Span on the South End of the Auxiliary Building between column lines S_1 and Q_1	N7	Maximum Column Load
	N8	Maximum Moment
	N9	Maximum Shear

11.4. Crane Sliding

Sliding of the crane wheels will not be considered and the boundary conditions for the crane are consistent with Table 5 in Section 11.1. This is consistent with the design input provided by P&H Morris Material Handling.

12. EVALUATION

12.1. Member Code Check

The steel members of the developed steel model will be evaluated by the GT STRUDL code checking function and manual calculations, if necessary. Member modifications will be made, if necessary, to qualify the Auxiliary Building for the upgraded crane.

12.2. Connection Evaluation

The loads at the member connections, from the building computer analysis will be compared to the original design loads. Connections experiencing loads in excess of the original design loads will be evaluated. Structural modifications will be designed where existing design is inadequate for the revised loads. The building anchorages to the concrete structure will be similarly evaluated and modified, if required.

13. ANSYS MODEL AND ACCEPTANCE CRITERIA

13.1. Model Development

An ANSYS model of the Auxiliary Building steel structure will be developed based on the GT STRUDL model. The ANSYS model of the Auxiliary Building steel structure will require an additional calculation to be performed to document a comparison of the dynamic response of the GT STRUDL and ANSYS analysis models.

The completed ANSYS model shall then be transmitted to the crane vendor by Progress Energy. The crane vendor shall use the ANSYS building model in conjunction with the crane model to perform a coupled building/crane dynamic analysis for qualification of the crane. Qualification/evaluation of the building portions of the coupled model will not be the responsibility of the crane vendor.

13.2. Acceptance Criteria

The GT STRUDL and ANSYS model must demonstrate a reasonable level of similarity in order to ensure compatibility between the building qualification and the design of the crane. The following checks will be performed.

13.2.1. Application of Unit Loading

As both the GT STRUDL and ANSYS models will be constructed with simple beam elements, the corresponding stiffness of the analytical models shall be compared through the application of concentrated unit loads. Identical concentrated unit loads will be applied at various points in each of the principal directions of the GT STRUDL and ANSYS structural models. The displacements and reactions due to the concentrated loads will be compared to ensure compatibility of the two models.

13.2.2. Application of Unit Accelerations

After the stiffness properties of the GT STRUDL and ANSYS models have been confirmed to be matching, it will be necessary to compare the mass properties of the two models. This will be achieved through the application of concentrated unit accelerations in each of the principal directions. The displacements and reactions due to the concentrated unit accelerations shall be compared to ensure compatibility of the two models.

13.2.3. Modal Frequencies and Mode Shapes

Once the unit load and unit acceleration tests have been successfully conducted, the modal responses for those frequencies that show significant excitation of the crane structure or building structural components in the proximity of the crane should be compared to ensure compatibility of the two models.

13.2.4. Mass Participation Factors

The mass participation factors of the two models below the cutoff frequency of 33 Hz will be compared using engineering judgment to determine if the models adequately demonstrate similarities within the critical frequency ranges. The critical frequencies would be those that produce significant responses within the crane structure and/or building structure in the proximity of the crane.

14. <u>REFERENCES</u>

14.1. Site Specifications and Procedures

14.1.1. OP-421C, Rev. 33, Operation of the Auxiliary Building Overhead Crane FHCR-5

14.1.2. Crystal River Nuclear Unit 3 Final Safety Analysis Report, Rev. 32

14.1.3. Design Basis Document 1/3, Rev. 6, Major Class I Structures

14.1.4. SP-5209, Rev. 0, CR3 Seismic Qualification,

14.1.5. SP-5757, Rev. 0, Specification for Erection of Structural Steel

14.1.6. RO-2968, Requirement Outline for Fabrication of Structural Steel

14.1.7. EGR-NGGC-0352, Rev. 5, Base Plate Design,

14.1.8. Environmental Qualification Plant Profile Document (EQPPD), Rev. 18

14.1.9. Design Basis Document 1/5, Rev. 3, Major Class III Structures

14.2. Industrial Codes, Standards, and Manuals

14.2.1. ASCE paper No. 3269, 1961, Wind Forces on Structures

14.2.2. AISC 6^{th} Edition, Manual of Steel Construction, 1963

14.2.3. ACI 318-63, Building Code Requirements for Reinforced Concrete

14.2.4. ACI 318-71, Building Code Requirements for Reinforced Concrete

14.2.5. NUREG-0554, Single-Failure-Proof Cranes for Nuclear Power Plants, May 1979

14.2.6. NUREG-0612, Control of Heavy Loads at Nuclear Power Plants, July 1980

14.2.7. USNRC Regulatory Guide 1.61, Damping Values for Seismic Design of Nuclear Power Plants

14.2.8. USNRC Regulatory Guide 1.92, Combining Modal Responses and Spatial Components in Seismic Response Analysis, Rev. 2, July 2006

14.2.9. ASME NOG-1, Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder), 2004

14.2.10. ASCE 7-05, Minimum Design Loads for Buildings and Other Structures.

14.2.11. NUREG-0800, Rev. 3, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, March 2007

14.2.12. AISC 9th Edition, Manual of Steel Construction, 1989

14.3. Calculations

14.3.1. Calculation 2:01.7D, Applied Load from Steel Structure

14.3.2. Calculation 2:01.10, Steel Frames

14.3.3. Calculation 2:01.11, Steel Columns

14.3.4. Calculation 2.01.12, Vertical Bracing

14.3.5. Calculation 2.01.13, Crane Runway Beams

14.3.6. Calculation 2:01.14, Steel Floor Framing @ EL. 162’-0”

14.3.7. Calculation 2:01.15, Roof Framing, Girts, and Miscellaneous Steel

14.3.8. Calculation 2:01.16, Seismic Analysis of Steel Frame

14.3.9. Calculation 2:01.48, Basic Design Requirements – Aux Bldg

14.3.10. Calculation 2:01.50, Structural Steel – Aux Bldg

14.3.11. Calculation 2:01.55, Support Walls and Columns – Aux Bldg

14.4. Other References

14.4.1. GT STRUDL Computer Program, User Manual, Georgia Institute of Technology, Version 30.0 (see Note below)

14.4.2. ANSYS Version 11 (see Note below)

14.4.3. R88752 Sh. 1, 2 & 3, Crane Layout 130 Ton SFP, Rev. 0 (DRAFT)

Note: GT STRUDL and ANSYS are commercially available computer software that is procured and maintained under ENERCON Services QA program

14.5. Drawings and Sketches

14.5.1. U-62238, General Arrangement of a Three Motor Tiger Trolley, Rev. A

14.5.2. 001-012, Layout - Plan above Reactor Auxiliary and Intermediate Buildings - Basement Floor - ELEV. 75’-0” and 95’-0”, Rev. 41 (001-012-SH000)

14.5.3. 001-022, Layout - Plan above Reactor Auxiliary and Intermediate Buildings - Mezzanine Floor ELEV. 119”-0”, Rev. 44 (001-022-SH000)

14.5.4. 001-023, Layout - Plan above Reactor Auxiliary and Intermediate Buildings - ELEV. 143”-0”, Rev. 26 (001-023-SH000)

14.5.5. 001-032, Layout - Plan above Reactor Building Floor Elev. 160’-0” & Auxiliary Building - ELEV. 162”-0”, Rev. 31 (001-032-SH000)

14.5.6. 001-042, Layout - Plan above Reactor - ELEV. 180”-0”, Rev. 12 (001-042-SH000)

14.5.7. 002-002, Layout - Cross Section Thru Reactor Bldg. & Auxiliary Building, Rev. 6 (002-002-SH000)

14.5.8. 002-003, Layout - Longitudinal Section Thru Reactor Bldg. & Spent Fuel Pit, Rev. 5 (002-003-SH000)

14.5.9. 201-304, Arrangement - Electrical Equipment - Reactor, Auxiliary & Intermediate Building EL. 143’-0”, Rev. 6 (201-304-SH000)

14.5.10. 201-305, Arrangement - Electrical Equipment - Reactor Building Operating Floor & Auxiliary Building EL. 162’-0”, Rev. 7 (201-305-SH000)

14.5.11. 216-100, EQ Environmental Zone Map - Reactor Building and Auxiliary Building EL. 95’-0”, Rev. 15 (216-100--EZ-001-SH000)

14.5.12. 216-100, EQ Environmental Zone Map - Reactor Building, Auxiliary Building and Intermediate Building EL. 119’-0”, Rev. 8 (216-100--EZ-002-SH000)

14.5.13. 216-100, EQ Environmental Zone Map - Reactor Building, Auxiliary Building and Intermediate Building EL. 143’-0”, Rev. 8 (216-100--EZ-003-SH000)

14.5.14. 216-100, EQ Environmental Zone Map - Reactor Building EL. 160’-0” - Auxiliary Building EL. 162’-0”, Rev. 8 (216-100--EZ-004-SH000)

14.5.15. 216-100, EQ Environmental Zone Map - Cross Section Thru Reactor Building and Auxiliary Building, Rev. 6 (216-100--EZ-007-SH000)

14.5.16. 216-100 NOD-001, EQ Node/Zone Map - Reactor Building and Auxiliary Building EL. 95’-0”, Rev. 1 (216-100--NOD-001-SH001)

14.5.17. 216-100 NOD-002, EQ Node/Zone Map - Plan EL. 119’-0”, Rev. 1 (216-100--NOD-002-SH001)

14.5.18. 311-715, Auxiliary Building - South End - Plan at EL. 143’-0”, Rev. 27 (311-715-SH000)

14.5.19. 311-716, Auxiliary Building & Control Complex - Plan at Floor EL. 143’-0” and 145’-8”, Rev. 23 (311-716-SH000)

14.5.20. 311-718, Auxiliary Building & Control Complex - Plan at Floor EL. 162’-0” and 164’-0”, Rev. 30 (311-718-SH000)

14.5.21. 311-720, Auxiliary Building and Control Building Sections, Rev. 20 (311-720-SH000)

14.5.22. 311-721, Auxiliary Building and Control Building Sections, Rev. 28 (311-721-SH000)

14.5.23. 311-722, Auxiliary Building and Control Building Sections, Rev. 33 (311-722-SH001-SH000)

14.5.24. 311-722, Auxiliary Building and Control Building Sections, Rev. 2 (311-722--SH002-SH000)

14.5.25. 311-818, Aux Bldg Sys Damper & Flow Monitor Support Details - Intermediate Steel Details, Rev. 2 (311-818--SH010-SH000)

14.5.26. 421-106, Auxiliary Building - Walls from EL. 93’-0” to EL. 119’-0”, Rev. 1 (421-106-SH000)

14.5.27. 421-107, Auxiliary Building - Walls from EL. 93’-0” to EL. 119’-0” Elevations & Sections, Rev. 12 (421-107-SH000)

14.5.28. 421-108, Auxiliary Building - Walls from EL. 93’-0” to EL. 119’-0” Elevations & Sections, Rev. 9 (421-108-SH000)

14.5.29. 421-110, Auxiliary Building North - Floor EL. 119’-0” - Plan Concrete Outline, Rev. 14 (421-110-SH000)

14.5.30. 421-111, Auxiliary Building North - Floor EL. 119’-0” - Plan Concrete Sec. and Details, Rev. 2 (421-111-SH000)

14.5.31. 421-113, Auxiliary Building North - Floor EL. 119’-0” - Sections & Details, Rev. 5 (421-113-SH000)

14.5.32. 421-114, Auxiliary Building North - Floor EL. 119’-0” - Sections & Details, Rev. 7 (421-114-SH000)

14.5.33. 421-115, Auxiliary Building North – Walls from EL. 119’-0” to EL. 143’-0” Plan, Rev. 3 (421-115-SH000)

14.5.34. 421-116, Auxiliary Building North - Walls from EL. 119’-0” to EL. 143’-0” Elevations & Sections, Rev. 6 (421-116-SH000)

14.5.35. 421-117, Auxiliary Building North - Walls from EL. 119’-0” to EL. 143’-0” Elevations & Sections, Rev. 7 (421-117-SH000)

14.5.36. 421-118, Auxiliary Building Plans, Rev. 1 (421-117-SH000)

14.5.37. 421-119, Auxiliary Building North - Floor EL. 143’-0” - Plan Concrete Outline, Rev. 8 (421-119-SH000)

14.5.38. 421-120, Auxiliary Building North - Floor EL. 143’-0” - Plan Anchor Bolts and Dowels, Rev. 1 (421-120-SH000)

14.5.39. 421-121, Auxiliary Building North - Floor EL. 143’-0” - Plan Reinforcement, Rev. 1 (421-121-SH000)

14.5.40. 421-122, Auxiliary Building North - Floor EL. 143'-0" - Sections and Details, Rev. 1 (421-122-SH000)

14.5.41. 421-123, Auxiliary Building North - Floor EL. 143'-0" - Sections and Details, Rev. 6 (421-123-SH000)

14.5.42. 421-125, Auxiliary Building - Knock Out Panels at EL. 143'-0" - Plans & Sections, Rev. 0 (421-125-SH000)

14.5.43. 421-127, Auxiliary Building North - Floor Slab EL. 162'-0" - Plans & Sections, Rev. 6 (421-127-SH000)

14.5.44. 421-129, Auxiliary Building North - Walls from EL. 143'-0" to EL. 162'-0" Plan, Rev. 4 (421-129-SH000)

14.5.45. 421-130, Auxiliary Building North - Walls from EL. 143'-0" to EL. 162'-0" Sections and Details, Rev. 6 (421-130-SH000)

14.5.46. 421-131, Auxiliary Building North - Walls from EL. 143'-0" to EL. 162'-0" Sections and Details, Rev. 4 (421-131-SH000)

14.5.47. 421-132, Auxiliary Building - Removable Hatch Covers, Rev. 0 (421-132-SH000)

14.5.48. 421-138, Auxiliary Building North - Penetration Closure Details - Walls above EL. 95'-0", Rev. 0 (421-132-SH000)

14.5.49. 421-139, Auxiliary Building North - Floor EL. 119'-0" - Penetration Closure Details, Rev. 5 (421-139-SH000)

14.5.50. 421-140, Auxiliary Building North - Miscellaneous Wall Elevations Penetration Closure Details, Rev. 3 (421-140-SH000)

14.5.51. 421-141, Auxiliary Building North - Spent Fuel Pit - Concrete Outline Plans & Sections, Rev. 13 (421-141-SH000)

14.5.52. 421-142, Auxiliary Building North - Spent Fuel Pit - Reinforcement - Plans & Sections, Rev. 3 (421-142-SH000)

14.5.53. 421-143, Auxiliary Building North - Spent Fuel Pit - Sections and Details, Rev. 6 (421-143-SH000)

14.5.54. 421-150, Auxiliary Building North - Floor EL. 95'-0", Penetration Closure Details, Rev. 1 (421-150-SH000)

14.5.55. 422-005, Auxiliary Building South - Foundation Mat EL. 93'-0", Plan Concrete Outline, Rev. 7 (422-005-SH000)

14.5.56. 422-010, Auxiliary Building South - Floor EL. 119'-0", Plan Concrete Outline, Rev. 21 (422-010-SH000)

14.5.57. 422-015, Auxiliary Building South - Walls from EL. 93'-0" to EL. 119'-0" Plan, Rev. 15 (422-015-SH000)

14.5.58. 422-019, Auxiliary Building South - Walls from EL. 119'-0" to EL. 143'-0" Plan, Rev. 8 (422-019-SH000)

14.5.59. 422-023, Auxiliary Building South - Floor EL. 143'-0", Plan Concrete Outline, Rev. 11 (422-023-SH000)

14.5.60. 422-031, Auxiliary Building South - Floor Slab EL. 162'-0", Plan Section & Details, Rev. 4 (422-031-SH000)

14.5.61. 521-101, Auxiliary Building North - Steel Framing Platform at EL. 131'-0" and 165'-5½" and Stairs, Rev. 5 (521-101-SH000)

14.5.62. 521-102, Auxiliary Building North - Steel Framing - Roof Steel - Plan Crane Runway - Roof EL 200'-4" & 209'-1", Rev. 6 (521-102-SH000)

14.5.63. 521-103, Auxiliary Building North & South - Miscellaneous Platforms & Monorails, Rev. 8 (521-103-SH000)

14.5.64. 521-105, Miscellaneous Steel - Intermediate & Auxiliary Building - Motor Control Cabinet Frames, Rev. 5 (521-105-SH000)

14.5.65. 521-107, Auxiliary 80' Long North - Miscellaneous Steel - Decontamination Pit Platforms and Ladder, Rev. 2 (521-107-SH000)

14.5.66. 521-109, Auxiliary Building - Missile Shield Crane Anchor, Rev. 1 (521-109-SH000)

14.5.67. 521-110, Auxiliary Building - Spent Fuel Pit - Plan & Sections, Rev. 10 (521-110-SH000)

14.5.68. 521-111, Auxiliary Building - Spent Fuel Pit Liner Plate - Sections, Rev. 5 (521-111-SH000)

14.5.69. 521-112, Auxiliary Building - Spent Fuel Pit Liner Plate - Sections & Details, Rev. 9 (521-112-SH000)

14.5.70. 522-001, Auxiliary Building - Steel Framing - Column Schedule, Rev. 1 (522-001-SH000)

14.5.71. 522-002, Auxiliary Building South Steel Framing - Platform EL. 119'-0" - Stairs and Typical Handrail and Toe Plate Details, Rev. 1 (522-002-SH000)

14.5.72. 522-003, Auxiliary Building South Steel Framing - Roof at EL. 167'-6" & Floor at EL. 162'-0", Rev. 6 (522-003-SH000)

14.5.73. 522-004, Auxiliary Building South Steel Framing - Roof at EL. 209'-1" & Crane Runway Steel at EL. 193'-7", Rev. 4 (522-004-SH000)

14.5.74. 522-006, Auxiliary Building South Steel Framing - Column Bracing, Rev. 3 (522-006-SH000)

14.5.75. 522-007, Auxiliary Building Steel Framing - East. South & West Girt Elevations, Rev. 1 (522-007-SH000)

14.5.76. 522-008, Auxiliary Building Steel Framing - West & South Girt Elevations, Rev. 1 (522-008-SH000)

14.5.77. 522-012, Auxiliary Building South - Misc. Steel - Roof Steel Framing - EL. 167'-6" @ Buttress No. 5, Rev. 0 (522-012-SH000)

Appendix 1

Design Methodology Flow Chart

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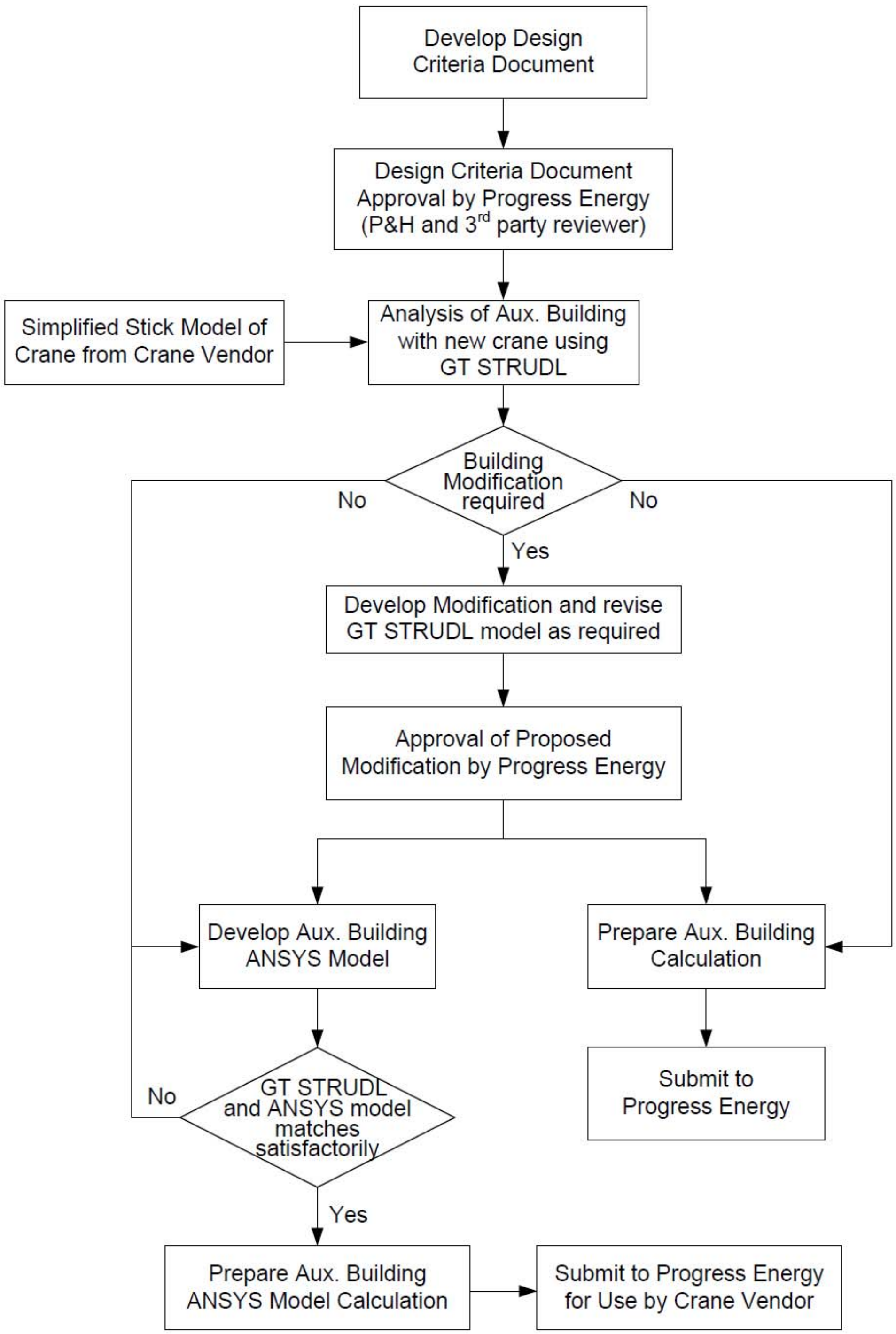
		Rev. 1
		Page No. A1.2



Appendix 2

Envelope Response Spectra for Seismic Evaluation

1.0 Objectives

The objective of this document is to:

- Generate response spectra (RS) for the Auxiliary Building at Elevation 162' for use in the seismic qualification of the steel structure. The scope of this work includes the determination of new spectra for:
  - Maximum Hypothetical Earthquake (MHE[1]) condition
  - Operating Basis Earthquake (OBE) condition
- Explain and justify the methodology used.

2.0 Introduction

Per Gilbert Calculation 2:01.10 (Ref. 3), the existing Auxiliary Building steel structure has been qualified using ground response spectra (GRS). However, in order to appropriately determine the effect of seismic loading at the level of the crane girder, the response of the concrete structure underlying the steel support structure should be considered. This is accomplished by incorporating the floor response spectra (FRS) at the concrete top elevation (162') into the input spectra for the coupled evaluation of the steel structure and crane system. Response spectra are thus obtained which envelope the appropriate GRS and FRS curves. In this way, a coupled analysis can be performed in which the structure and crane are evaluated in a consistent manner and within the original design basis.

The GRS curves are defined in the FSAR (Ref. 1), both in terms of the 0.05g OBE condition (Figure 2-35) and the 0.10g MHE condition (Figure 2-36). Based on Section 5.2.4.1.2 of the FSAR (Ref. 1), the damping value to be used during seismic analysis of bolted steel structures is 2.5% of critical. However, the OBE GRS curve at 1% damping was used for the original seismic analysis of the steel structure based on Gilbert calculation (Refs. 3 and 4). Thus the 1% damping curve is chosen for the MHE GRS curve, which is verified to be more conservative than the 2.5% damping curve (see Section 5.0). The 2.5% damping curve is interpolated from the reported 2% and 5% curves (see Section 3.1 for methodology).

[1] "MHE" is the CR-3 site-specific term for Safe Shutdown Earthquake (SSE).

The FRS curves are defined based on those previously calculated. FRS for Auxiliary Building elevations up to 162’ had been developed initially for equipment damping values of 0.5% and 1.0% of critical in Reference 5 using the methodology of Reference 6. Later, floor response spectra for elevations above 162’ were developed and are reported in Reference 7. These higher elevations represent those of the steel structure on top of the concrete building. MHE spectra were then developed for equipment damping values of 2%, 3%, and 5% of critical in Reference 8 which were modified from the earlier spectra having 0.5% and 1% damping. As per ASME NOG-1 (Ref. 9) Section 4153.8, crane design should be performed using damping values of 4% for an OBE condition and 7% for an MHE condition. Therefore, the damping value for the FRS portion of the developed RS curves is considered to be 7% for the MHE condition, and 4% for the OBE condition. Per Reference 7, OBE spectra can be taken as half of MHE spectra. Therefore, the OBE FRS for 4% damping will be a linear interpolation of the MHE FRS curves for 3% and 5% damping with amplitudes divided by two. Because no FRS is available with damping greater than 5%, interpolation cannot be used to obtain an MHE FRS for 7% damping. Instead, damping modification methods will be used.

3.0 Methodology

The desired acceleration response spectrum is a function of vibration period (T=1/f where f=vibration frequency) and damping ratio (ξ), and is here defined as:

$$S_A(T,\xi)$$

3.1 Interpolation

Interpolation may be used when the system in question has a damping ratio between two damping ratios with associated design spectra (ξ_l, ξ_h), such that

$$\xi_1 < \xi < \xi_h$$

In this case, the desired acceleration response spectra can be described by a simple algebraic function of the spectra at the lower and higher damping ratios:

$$S_A(T,\xi) = S_A(T,\xi_1) - \frac{(\xi_1 - \xi)}{(\xi_1 - \xi_h)}(S_A(T,\xi_1) - S_A(T,\xi_h))$$

3.2 Damping Correction Factors

There are several well-understood and common methods for modifying existing response spectra to estimate corresponding spectra with different damping. Four methods are considered for use here, followed by a brief comparison to choose the most conservative under the specific conditions.

3.2.1 Power Method

The Power method represents an analytical methodology for determining response spectra for a given damping ratio, ξ, given the response spectra of two different damping ratios, ξ_1 and ξ_2. It is based on structural dynamics and general vibration theory, and is derived in Reference 8. The governing equation is a power law relationship, and can be written as:

$$S_A(T,\xi) = S_A(T,\xi_1)^{1-\varepsilon} \cdot S_A(T,\xi_2)^{\varepsilon},\ \varepsilon = \frac{\ln(\xi/\xi_1)}{\ln(\xi_2/\xi_1)}$$

The response spectrum resulting from the Power Method is shown in Figure A2.1 below.

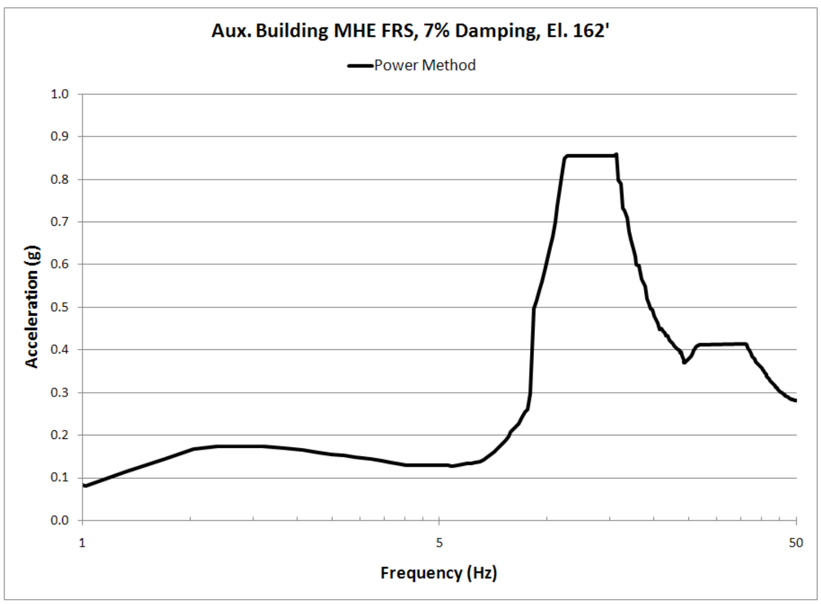


Figure A2.1: Response spectrum generated by the Power method.

3.2.2 Newmark and Hall

Perhaps the most well-known of the damping modification methods is the Newmark and Hall method (Ref. 11), which has been adopted in several building codes and structural guidance documents. Based on the results of analysis of a number of systems to a range of earthquakes recorded prior to 1973, empirical spectrum amplification factors were defined, which are used to multiply the peak ground response to determine a median estimate of the elastic response at a given damping, ξ. These amplification factors are defined differently in the constant acceleration, velocity, and displacement frequency range, and are dependent on the damping ratio (expressed as %, not decimal form):

$3.21 - 0.68 \ln \xi$ constant acceleration region

$2.31 - 0.41 \ln \xi$ constant velocity region

$1.82 - 0.27 \ln \xi$ constant displacement region

Given a response spectra evaluated for a specific damping value, a ratio of spectra amplification factors can be used to determine the response spectra at a different damping value. The spectra amplification factors are often reported as damping reduction factors, which are simply spectra amplification factor ratios between the desired damping and a standard damping value, typically 5%.

The response spectrum resulting from the Newmark and Hall Method is shown in Figure A2.2 below.

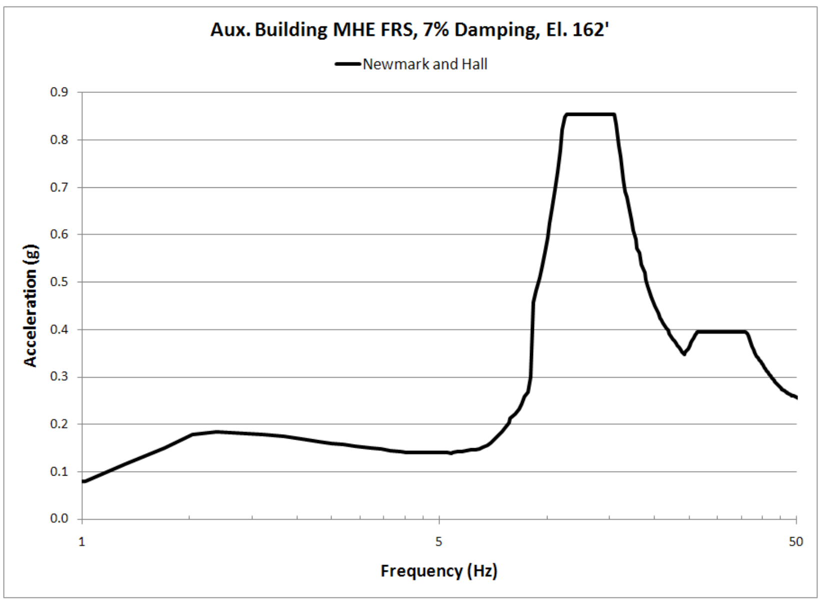


Figure A2.2: Response spectrum generated by the Newmark and Hall method.

3.2.3 Lin and Chang

In the Lin and Chang method (Ref. 12), a damping reduction factor (B) adjusts the known spectra at 5% damping to an estimated spectra at higher damping (ξ) such that:

$$S_A(T, \xi) = B(T, \xi) \times S_A(T, 5\%)$$

The damping reduction factor, dependent on vibration period and damping ratio (expressed in decimal form, not %) is defined as:

$$B(T, \xi) = 1 - \frac{aT^{0.30}}{(T+1)^{0.65}}, \qquad a = 1.303 + 0.436 \ln(\xi)$$

Similar to the Newmark and Hall method, the damping reduction factor in the Lin and Chang method is empirically based on analysis of numerous systems to a range of earthquakes. However, the Lin and Chang method uses a much broader range of system characteristics and a much larger and more diverse library of acceleration time histories.

The response spectrum resulting from the Lin and Chang Method is shown in Figure A2.3 below.

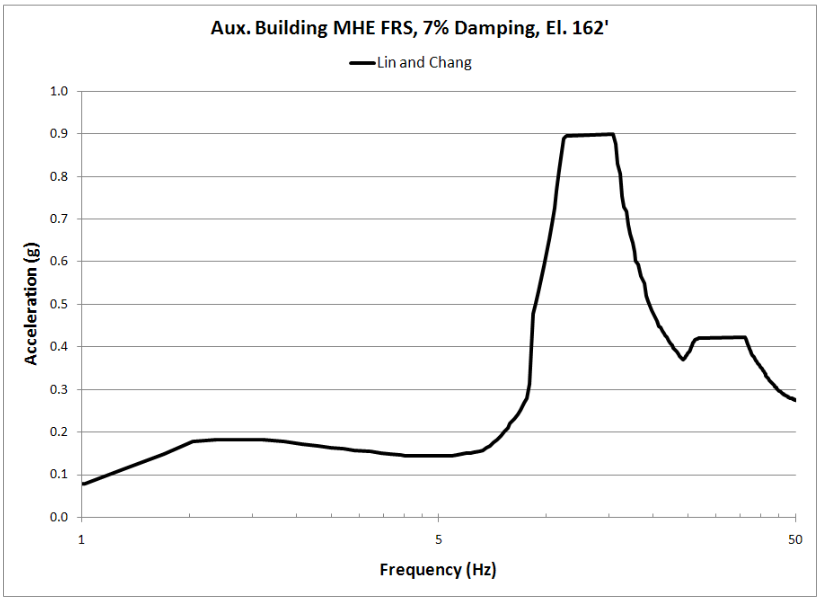


Figure A2.3 Response spectrum generated by the Lin and Chang method.

3.2.4 General Implementation Procedure (GIP)

The Seismic Qualification Utility Group (SQUG) prepared a General Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment (Ref. 13) which endorsed a method for obtaining in-structure response spectra at different damping levels than those already available. The method is based on Appendix A of Reference 14. In this method, the in-structure response spectra at some desired damping ratio ξ_D is determined based on the spectra defined at damping ratio ξ_A, such that:

$$S_A(T,\xi_D) = S_A(T,\xi_A)\sqrt{\frac{\xi_A}{\xi_D}}$$

The response spectrum resulting from the GIP Method is shown in Figure A2.4 below.

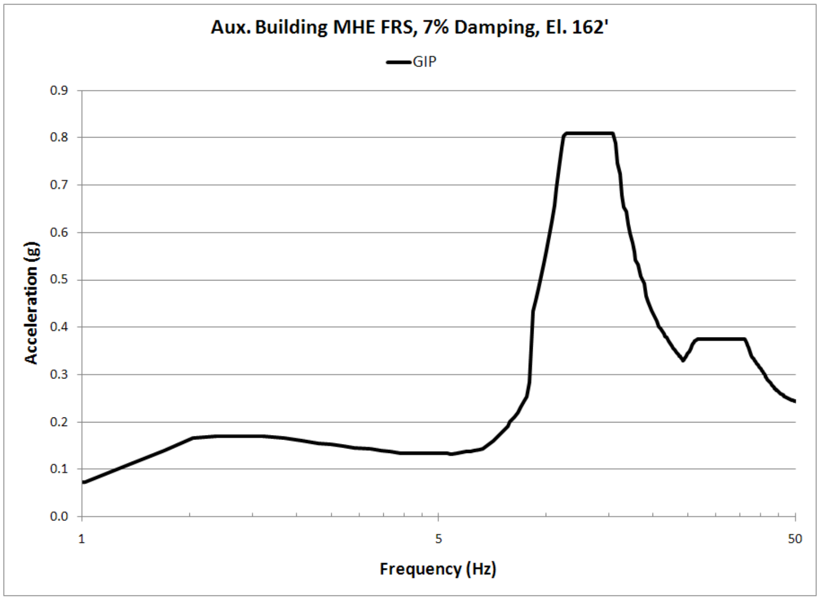


Figure A2.4 Response spectrum generated by the GIP method.

3.2.5 Comparison

The four methods above are used to estimate an MHE FRS curve for elevation 162' of the Auxiliary Building for a damping value of 7% of critical, as depicted in Figure A2.5 below. The power method uses the MHE FRS at 3% and 5% damping reported in Reference 8 as input spectra. The Newmark and Hall, Lin and Chang, and GIP methods use the 5% damping MHE FRS as input. The spectra amplification factors of the Newmark method were calculated based on a range of constant acceleration ≥ 1.5 Hz, constant displacement ≤ 0.243 Hz, and constant velocity elsewhere, from Reference 15.

In general, the spectra produced by the four discussed methods are reasonably close. The NRC-endorsed GIP method provides the least conservative result, and is therefore not used. The Newmark and Hall method has been shown in Reference 16 to underestimate response where vibration period is less than 0.2 s (frequency greater than 5 Hz). This effect is seen here. In addition, the power method also predicts a lower response than the Lin and Chang method at the main peak (approximately 12-15 Hz). This can be troublesome in the case of FRS, where spectra will be used to evaluate response of mounted equipment that is likely to have short vibration periods. Therefore, the Lin and Chang method is used here, which provides more conservative estimates of response in the high frequency range than Newmark and Hall without reduced accuracy elsewhere in the spectrum (Ref. 16).

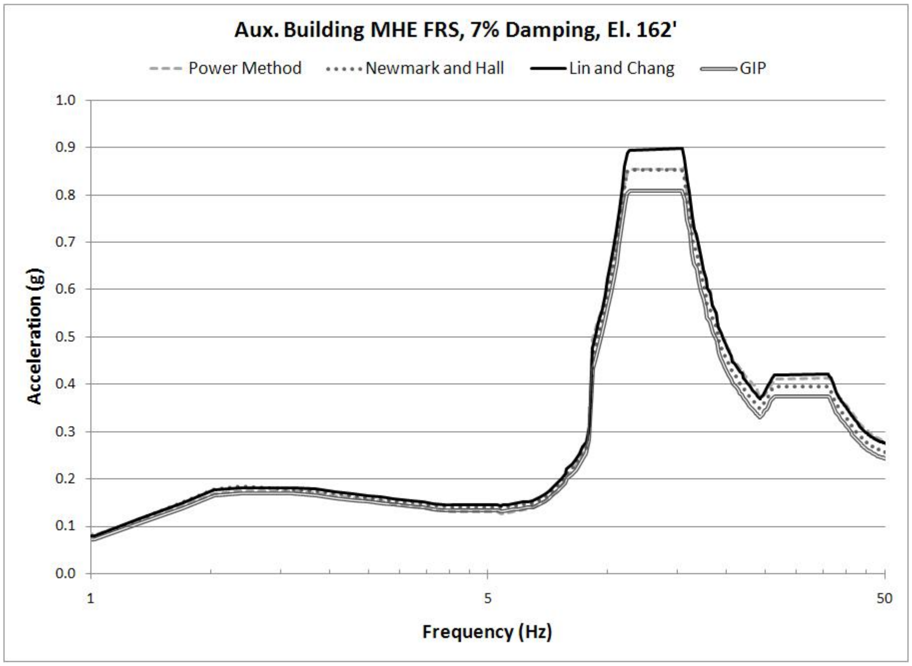


Figure A2.5 Response spectrum generated by the GIP method.

3.3 Envelope Spectra

As discussed in Section 2.0, the response spectra obtained for use in the coupled evaluation of steel structure and supported crane will incorporate the appropriate GRS and FRS curves. This is accomplished by utilizing envelope curves, resulting in response spectra which are more conservative than the utilizing the GRS alone. The envelope spectra are thus defined as the greater of the appropriate (OBE or MHE) GRS or modified (to account for damping) FRS at each frequency. Where FRS with 7% damping are used, the Lin and Chang method was used to compute the curves. Because the lowest reported frequency of the GRS curves in the FSAR (Ref. 1) is 1 Hz, the new FRS curves define the envelope response spectra at frequencies below 1 Hz.

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4.0 Envelope Response Spectra, OBE, Elevation 162'

As described in Section 2.0, the Auxiliary Building response spectrum at elevation 162' for the OBE condition is an envelope spectra made up of the OBE GRS curve at 1% damping and the elevation 162' FRS curve at 4% damping, as shown in Figure A2.6 below. The 4% damping OBE FRS curve is determined based on a linear interpolation of the existing MHE FRS curves with 3% and 5% damping, with the amplitude divided by two. The OBE GRS spectrum is defined at 1% damping in the FSAR (Ref. 1) Figure 2-35. These two spectra cross at 7.47 Hz; thus, the envelope response spectra is determined by the OBE GRS 1% curve for frequencies between 1 Hz and 7.47 Hz, and the interpolated and modified FRS 4% curve at frequencies greater than 7.47 Hz and less than 1 Hz. The resulting response spectra is illustrated and tabulated below.

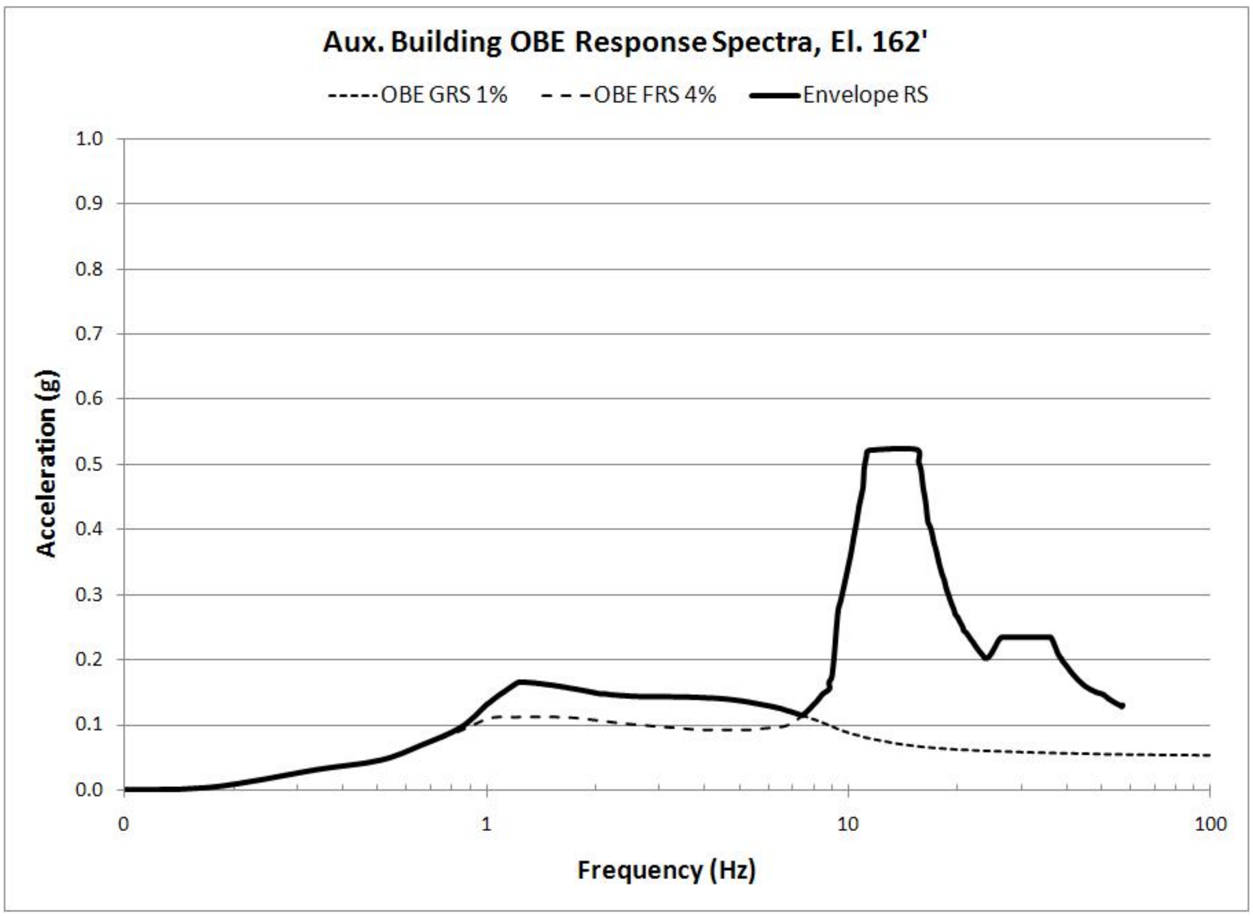

Figure A2.6 Auxiliary Building OBE Response Spectra @ EL. 162'-0"

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Auxiliary Building OBE Response Spectra, EL. 162'

Freq. (Hz)	Acc. (g)	Freq. (Hz)	Acc. (g)
0.09	0	5.75	0.131
0.17	0.003	6.00	0.129
0.34	0.031	6.25	0.127
0.51	0.046	6.50	0.125
0.66	0.068	6.75	0.121
0.85	0.096	7.00	0.119
1.00	0.132	7.25	0.116
1.10	0.148	7.50	0.114
1.20	0.163	7.75	0.122
1.30	0.165	8.00	0.130
1.40	0.163	8.50	0.148
1.50	0.161	9.00	0.175
1.60	0.159	9.50	0.290
1.70	0.156	10.0	0.348
1.80	0.154	10.5	0.410
1.90	0.152	11.0	0.465
2.00	0.149	11.5	0.523
2.10	0.147	12.0	0.523
2.20	0.146	12.5	0.523
2.30	0.145	13.0	0.523
2.40	0.144	13.5	0.523
2.50	0.144	14.0	0.523
2.60	0.143	14.5	0.523
2.70	0.143	15.0	0.523
2.80	0.143	15.5	0.523
2.90	0.143	16.0	0.470
3.00	0.143	17.0	0.395
3.15	0.143	18.0	0.339
3.30	0.143	20.0	0.266
3.45	0.143	22.0	0.228
3.60	0.143	23.5	0.207
3.80	0.142	25.0	0.212
4.00	0.142	26.0	0.235
4.20	0.141	28.0	0.235
4.40	0.140	31.0	0.235
4.60	0.140	34.0	0.235
4.80	0.138	36.0	0.235
5.00	0.137	40.0	0.191
5.25	0.135	45.0	0.159
5.50	0.133	50.0	0.148

5.0 Envelope Response Spectra, MHE, Elevation 162’

As described in Section 2.0, the Auxiliary Building response spectrum at elevation 162’ for the MHE condition is an envelope spectra made up of the MHE GRS curve at 1% damping and the elevation 162’ FRS curve at 7% damping, as shown in Figure A2.7 below. Note that the GRS 1% curve, defined in the FSAR (Ref. 1) Figure 2-36, is more conservative than the GRS 2.5% curve, evaluated as a linear interpolation between the GRS curves defined at 2% and 5% damping. The Lin and Chang method (Ref. 12) is used to modify the existing MHE FRS spectra with 5% damping to reflect 7% damping. These GRS and FRS spectra cross at 7.85 Hz; thus, the envelope response spectra is determined by the MHE GRS 1% curve for frequencies between 1 Hz and 7.85 Hz, and the interpolated and modified FRS 4% curve at frequencies greater than 7.85 Hz and less than 1 Hz. In the region around 7.85 Hz, some manual smoothing was performed. The resulting response spectra is illustrated and tabulated below.

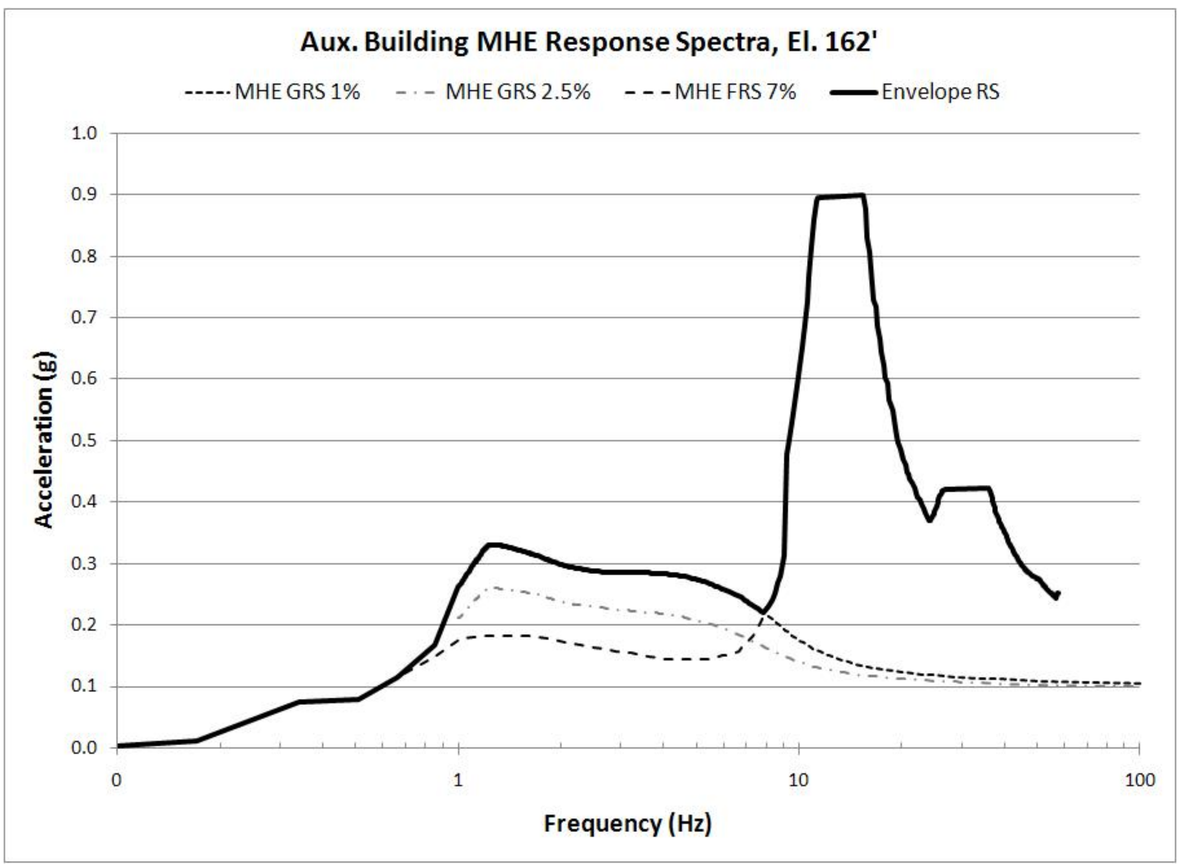

Figure A2.7 Auxiliary Building MHE Response Spectra @ EL. 162’-0”

Auxiliary Building MHE Response Spectra, EL. 162'

Freq. (Hz)	Acc. (g)	Freq. (Hz)	Acc. (g)
0.09	0	5.75	0.263
0.17	0.013	6.00	0.259
0.34	0.074	6.25	0.254
0.51	0.079	6.50	0.250
0.66	0.115	6.75	0.245
0.85	0.168	7.00	0.239
1.00	0.263	7.25	0.232
1.10	0.297	7.50	0.228
1.20	0.325	7.75	0.222
1.30	0.330	8.00	0.226
1.40	0.327	8.50	0.253
1.50	0.322	9.00	0.312
1.60	0.318	9.50	0.531
1.70	0.313	10.0	0.622
1.80	0.308	10.5	0.720
1.90	0.302	11.0	0.855
2.00	0.299	11.5	0.895
2.10	0.295	12.0	0.895
2.20	0.293	12.5	0.895
2.30	0.291	13.0	0.895
2.40	0.289	13.5	0.895
2.50	0.288	14.0	0.895
2.60	0.287	14.5	0.895
2.70	0.286	15.0	0.896
2.80	0.286	15.5	0.896
2.90	0.286	16.0	0.805
3.00	0.286	17.0	0.687
3.15	0.286	18.0	0.598
3.30	0.286	20.0	0.477
3.45	0.286	22.0	0.414
3.60	0.285	23.5	0.384
3.80	0.284	25.0	0.385
4.00	0.283	26.0	0.420
4.20	0.282	28.0	0.420
4.40	0.281	31.0	0.420
4.60	0.279	34.0	0.420
4.80	0.277	36.0	0.420
5.00	0.274	40.0	0.351
5.25	0.271	45.0	0.297
5.50	0.267	50.0	0.276

6.0 References

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7.0 Summary

Seismic Analysis for Aux. Building Steel Structure
Enveloped [1] Response Spectra

	Current Licensing Basis [2]	ASME NOG-1-2004	Revised Aux. Building Qualification
Operating Basis Earthquake (OBE)	**FSAR:** OBE Ground Response Spectra (GRS) with damping value of 1% for welded and 2.5% for bolted structure (Ref. FSAR Section 5.2.4.1.2) **Analysis:** OBE Ground Response Spectra (GRS) with 1% damping (Ref. Gilbert Calculations 2:01)	Applicable OBE Response Spectra for the CR-3 site at appropriate level with 4% damping (Ref. ASME NOG-1-2004, Section 4152 & 4153.8)	**OBE** Spectra envelopes: • Current Licensing Basis • OBE Floor Response Spectra (FRS) at EL. 162' with 4% damping [3] NOTE: The enveloped response spectra conservatively envelopes both the current licensing basis & ASME NOG-1 requirement.
Maximum Hypothetical Earthquake (MHE)	**FSAR:** MHE Ground Response Spectra (GRS) with damping value of 1% for welded and 2.5% for bolted structure (Ref. FSAR Section 5.2.4.1.2) **Analysis:** MHE not included	Applicable MHE Response Spectra for the CR-3 site at appropriate level with 7% damping (Ref. ASME NOG-1-2004, Section 4152 & 4153.8)	MHE Spectra envelopes: • Current Licensing Basis • MHE Floor Response Spectra (FRS) at EL. 162' with 7% damping [3] NOTE: The enveloped response spectra conservatively envelopes both the current licensing basis & ASME NOG-1 requirement.

1) Enveloped spectra refers to a composite response spectra comprised of the maximum responses from each of the contributing response spectra.

2) GRS curves from FSAR, Fig. 2-35 for OBE (to a ground acceleration of 0.05 g acting horizontally and 0.033 g acting vertically) and Fig. 2-36 for MHE (to a ground acceleration of 0.1 g acting horizontally and 0.067 g acting vertically): Weston Geophysical Research, Inc., Seismicity Analysis and Response Spectra for Crystal River Nuclear Power Plant, June 27, **1967**.
NOTE: GRS curve for 2.5% damping is obtained using linear interpolation of the GRS curves for 2% and 5%, **2010**.

3) - OBE FRS curves for Aux. Building elevation up to 162' for damping values of 0.5% and 1% were developed in calculation S73-0001, Revision 0, "Response Spectrum Analysis", by M.P.H., **1973**.
- FRS curves for Aux. Building elevation for damping values of 2%, 3%, and 5% were developed in S92-0171, Revision 0, "Floor Response Spectrum Generation", by S.J. Serhan, **1992**.
OBE: FRS curve @ EL. 162' for 4% damping is obtained using linear interpolation of the OBE FRS curves for 3% and 5% damping, **2010**.
MHE: FRS curve @ EL. 162' for 7% damping is obtained using Lin and Chang method using MHE FRS curve for 5% damping, **2010**.
(NOTE: Lin & Chang method bounds Power, Newmark and Hall, and General Implementation Procedure (GIP) methods.)

Appendix 3

Comparison of Load Combinations

The load combinations for the steel structure shall be in accordance with the original Auxiliary Building Calculations and Section 4140 of ASME NOG-1. The following load combinations shown in Table A3.1 are used in the evaluation of the Auxiliary Building. The load combinations include the effect of dead and live load, the crane lifted load, crane impact loads specified by DBD 1/3, a design wind load, an operating wind load of 50 mph, and the MHE earthquake load. Tornado effects will not be considered in accordance with the original design calculations.

In order to satisfy the original licensing basis and the requirements of ASME NOG-1, the load combinations used in the analysis of the building shall be equal to or envelope the load combinations specified in ASME NOG-1 and the load combinations used in the original design calculations. The load combinations are shown to envelope the required load combinations in Table A3.2 below.

Table A3.1: Load Combinations used in the evaluation of the Auxiliary Building

Load Cases	GT STRUDL Load Combination
LC1	Dead Load + Live Load + Crane lift load
LC2	Dead Load + Live Load + Crane lift load + Vert. Impact
LC3	Dead Load + Live Load + Crane lift load + Trans. Impact
LC4	Dead Load + Live Load + Crane lift load + Long. Impact
LC5	Dead Load + Live Load + Crane lift load + Vert. Impact + Trans. Impact + Long Impact + Op. Wind
LC6	NOT USED
LC7	NOT USED
LC8	Dead Load + Live Load + Crane lift load + Design Wind
LC9	Dead Load + Live Load + Crane lift load + EQ (MHE)
LC10	Dead Load + Live Load + Crane lift load + EQ (MHE) + Op. Wind
LC11	Dead Load + Live Load + Crane lift load + EQ (OBE)
LC12	Dead Load + Live Load + Crane lift load + EQ (OBE) + Op. Wind
LC13	Dead Load + Live Load + Design Wind
LC14	Dead Load + Live Load + EQ (MHE) + Op. Wind
LC15	Dead Load + Live Load + EQ (OBE) + Op. Wind

Table A3.2: Load comb inations used in the ana lysis bound the load co mbinations required by ASME NOG-1 and the original calculations.

		Load Case	Bounding Load Case
Gilbert Calculations	GC1	Gravity + Vertical Impact	LC2
	GC2	Gravity + Horizontal Impact	LC3 & LC4
	GC3	Gravity + Wind	LC8
	GC4	Gravity + Seismic (OBE)	LC11
ASME NOG-1 Crane Operating Loads[2]	PC1	$P_{db} + P_{dt} + P_{lr}$	LC1
	PC2	$P_{db} + P_{dt} + P_{lr} + P_v + P_{wo}$	LC5
	PC3	$P_{db} + P_{dt} + P_{lr} + P_{ht} + P_{wo}$	LC5
	PC4	$P_{db} + P_{dt} + P_{lr} + P_{hl} + P_{wo}$	LC5
	PC5	$P_{db} + P_{dt} + (P_p \text{ or } P_{tp})$	N/A
ASME NOG-1 Construction Loads[2]	PC6	$P_{db} + P_{dt} + P_{cn} + P_v + P_{wo}$	LC5
	PC7	$P_{db} + P_{dt} + P_{cn} + P_{ht} + P_{wo}$	LC5
	PC8	$P_{db} + P_{dt} + P_{cn} + P_{hl} + P_{wo}$	LC5
ASME NOG-1 Severe Environmental Loads	PC9	$P_{db} + P_{dt} + P_{wd}$	LC13
ASME NOG-1 Extreme Environmental Loads	PC10	$P_{db} + P_{dt} + P_{cs} + P_{e'} + P_{wo}$	LC10
	PC11	$P_{db} + P_{dt} + P_{e'} + P_{wo}$	LC14
	PC12	$P_{db} + P_{dt} + P_{co} + P_e + P_{wo}$	LC12
	PC13	$P_{db} + P_{dt} + P_e + P_{wo}$	LC15
	PC14	$P_{db} + P_{dt} + P_{wt}$	N/A
ASME NOG-1 Abnormal Event Loads	PC15	$P_{db} + P_{dt} + P_a + P_{wo}$	N/A

P_{dt} = Trolley Dead Load
P_{db} = Bridge / Gantry Dead Load
P_{lr} = Rated Load
P_{lc} = Critical Load
P_{co} = Credible Critical Load with OBE
P_{cs} = Credible Critical Load with SSE[1]
P_{cn} = Construction Load
P_v = Vertical Impact Load
P_{ht} = Transverse Horizontal Load
P_{hl} = Longitudinal Horizontal Load
P_{wo} = Operating Wind Load
P_{wd} = Design Wind Load
P_{wt} = Tornado Wind Load
P_p, P_{tp} = Plant Operation Induced Loads
$P_{e'}$ = SSE Loads[1]
P_e = OBE Loads
P_a = Abnormal Event Loads

Note:

(1) SSE = MHE

(2) As simultaneous operation of motions is permitted, the impact loads shall be considered simultaneously as appropriate.

Appendix 4

Analysis Considerations
vs.
Current Licensing Basis / ASME NOG-1-2004

	CURRENT LICENSING BASIS	ASME NOG-1 REQUIREMENT	ANALYSIS CONSIDERATIONS
Analysis Methodology: Decoupled v. Coupled Analysis	The building was originally qualified using hand calculations, considering static seismic methodology. Gilbert Calculation 2:01.16 provides the seismic evaluation of the building using the crane wheel loads and masses provided by the original crane vendor.	In accordance with ASME NOG-1 Section 4153.5, since the total mass of the crane is large with respect to the mass of the runway system, it is required that a coupled analysis of the building and crane be conducted.	A coupled analysis incorporating the Auxiliary Building steel structure and a stick model of the crane, as provided by the crane vendor shall be included in the model used to qualify the building.
Response Spectrum Analysis (Damping Values): Ground Response Spectrum vs. Floor Response Spectrum	Gilbert Calculation 2:01.16 applies the ground response spectrum at the base of the steel structure. The OBE ground response spectrum with 1% damping is used in the Gilbert Calculations 2:01.10.	ASME NOG-1 Section 4152 states that the seismic input data shall be specified as response spectra at an appropriate level in the structure supporting the crane. ASME NOG-1 Section 4153.8 dictates that the crane design should be performed using damping values of 4% for an OBE condition and 7% for an MHE condition.	A 4% OBE and a 7% MHE FRS obtained and combined with the 1% GRS. For the detail of the response spectra, see Appendix 2 of this document. The enveloped response spectra includes the following: • FRS with 4% damping for OBE / 7% damping for MHE (ASME NOG-1) • GRS with 1% damping (FSAR)
Directional Combinations: Absolute Sum vs. SRSS	The FSAR 5.2.1.2.9 states that the respective vertical and horizontal seismic components at any point on the building shall be added by summing the absolute values of the response of each contributing frequency due to vertical motion to the corresponding absolute values of the response of each contributing frequency due to horizontal motion. (higher of N/S & Vert. or E/W & Vert.)	ASME NOG-1 Section 4153.10 states that the representative maximum values of the structural responses of each of the three-directional components of earthquake motion shall be combined by taking the square root of the sum of the squares of the maximum representative values of the co-directional responses caused by each of the three components of earthquake motion at each node of the crane mathematical model.	The resulting total responses in the structural members shall be the envelope of the absolute sum of the responses in the vertical and one horizontal direction (in accordance with the FSAR) with the SRSS combination of the responses in the three directions (in accordance with ASME NOG-1).
Tornado Wind / Tornado Missiles	The FSAR Section 5.1.1.1 states that the Auxiliary Building (excluding the steel roof support structure) is a Class I structure. The steel portion of the Auxiliary Building is not designed for tornado wind and tornado missiles, as per the Gilbert Calculations and FSAR Section 5.4.3.2.2. The Auxiliary Building steel structure was qualified for the design wind loads specified in FSAR.	ASME NOG-1 Section 4134 (c) states that tornado winds should be considered in the design of the crane. Tornado pressure differentials associated with the plant design basis tornado shall be included in the loading. Tornado-generated missiles shall be considered. Under these loadings, the crane will not be operational, but be secured. Indoor cranes may be subjected to the design basis tornado if the building enclosures have been designed to fail.	The qualification of the Auxiliary Building overhead crane (FHCR-5) supporting steel structure will stay consistent with the current licensing basis and will not consider tornado wind or tornado missiles.
Load Combinations: Earthquake Load & Operating Wind	The original qualification of the Auxiliary Building steel structure did not consider a load combination that takes an earthquake load in conjunction with an operating wind. The building was only qualified for the design wind speed in Gilbert Calculation 2:01.10.	ASME NOG-1 Section 4140 includes load combinations that combine the operating wind load with an earthquake load.	The load combinations specified in ASME NOG-1 that combines earthquake loads with an operating wind load shall be used. Present design basis does not provide any operating wind speed. Basic wind speed of 50 mph is considered as crane operating wind speed in the analysis.

	CURRENT LICENSING BASIS	ASME NOG-1 REQUIREMENT	ANALYSIS CONSIDERATIONS
Sliding	No Current Licensing Basis	NUREG-0554 Section 2.5 states that overhead cranes should be designed to remain in place on their respective runways with their wheels prevented from leaving the tracks during a seismic event. If a seismic event comparable to a safe shutdown earthquake (SSE) occurs, the bridge should remain on the runway with brakes applied, and the trolley should remain on the crane girders with brakes applied. ASME NOG-1 states that the crane must be able to stop and hold a critical load during a seismic event.	The Auxiliary Building steel structure will be qualified using a design methodology consistent with the crane vendor analysis methodology that does not consider sliding. The crane will be designed to meet the requirements of NUREG-0554 and ASME NOG-1 and sliding will not be considered.
Crane Impact Loads	The following impact factors are as stated in DBD 1/3: Longitudinal to the crane runway girder – 10% of maximum wheel load Transverse to the crane runway girder – 20% of trolley and lifted load Vertical – 25% of the lifted load	ASME NOG-1 Section 4133 states that the following impact factors will be used: Longitudinal to the crane runway girder – 5% of bridge dead load, trolley dead load, and maximum lift load Transverse to the crane runway girder – 10% of the trolley dead load and the maximum lift load Vertical – 15% of the maximum lifted load As per the boundary conditions specified by ASME NOG-1, the transverse loads are transmitted to only one crane runway girder.	The impact factors applied to the crane rails will take into consideration the factors specified in DBD 1/3 and ASME NOG-1. See Section 7.5 for the explanations the impact factors. Factors used: Longitudinal to the crane runway girder – DBD 1/3 Transverse to the crane runway girder – DBD 1/3; Vertical – DBD 1/3.