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UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

March 30, 2011

UN#11-116

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Subject: UniStar Nuclear Energy, NRC Docket No. 52-016
Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 253, Seismic System Analysis

References:
1) Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 253 SEB2 4788" email dated July 12, 2010

2) UniStar Nuclear Energy Letter UN#10-285, from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 253, Seismic System Analysis, dated November 16, 2010

The purpose of this letter is to respond to the request for additional information (RAI) identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated July 12, 2010 (Reference 1). This RAI addresses Seismic System Analysis, as discussed in Section 3.7 of the Final Safety Analysis Report (FSAR), as submitted in Part 2 of the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 Combined License Application (COLA), Revision 7.

Reference 2 anticipated that the response for RAI 268 Questions 03.07.02-45, 03.07.02-46, 03.07.02-49, 03.07.02-50 and 03.07.02-51 would be provided by March 31, 2011.

The Enclosure provides our response to RAI 253 Question 03.07.02-46 (partial, for the ESWB and EPGB), and Questions 03.07.02-49, 03.07.02-50 and 03.07.02-51, and includes revised

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COLA content. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

A response to Question 03.07.02-45 will be provided by April 29, 2011. The CWIS portion of Question 03.07.02-46 will be provided by May 27, 2011.

Our response does not include any new regulatory commitments. This letter does not contain any sensitive or proprietary information.

If there are any questions regarding this transmittal, please contact me at (410) 470-4205, or Mr. Wayne A. Massie at (410) 470-5503.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on March 30, 2011

Greg Gibson

Enclosure: Response to NRC Request for Additional Information, RAI 253 Question 03.07.02-46 for the ESWB and EPGB, and Questions 03.07.02-49, 03.07.02-50 and 03.07.02-51, Seismic System Analysis, Calvert Cliffs Nuclear Power Plant, Unit 3

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
Laura Quinn, NRC Environmental Project Manager, U.S. EPR COL Application
Getachew Tesfaye, NRC Project Manager, U.S. EPR DC Application (w/o enclosure)
Charles Casto, Deputy Regional Administrator, NRC Region II (w/o enclosure)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2
U.S. NRC Region I Office

Enclosure

Response to NRC Request for Additional Information

RAI 253 Question 03.07.02-46 for the ESWB and EPGB, and Questions 03.07.02-49, 03.07.02-50 and 03.07.02-51 Seismic System Analysis

Calvert Cliffs Nuclear Power Plant Unit 3

RAI 253

Question 03.07.02-46

The applicant in its response stated that the Switchgear Building (SB) will be analyzed using the same methodology as the Turbine Building (TB) which is described in the response to U.S. EPR FSAR NRC RAI 248, Question 03.07.02-56. In this methodology buildings are designed in such a way that the deformation, collapse, or partial collapse due to SSE loads are controlled by introducing an eccentrically braced frame in steel structures and a "crumple zone" in concrete structures. AREVA stated that this meets Acceptance Criteria 8A of SRP 3.7.2. The staff is still reviewing the AREVA response and has asked a follow-up question requesting additional information. With respect to the applicant's response to use the methodology proposed by AREVA, the applicant is requested to provide the following additional information for the TB and SB. As the Access Building (AB) is also the design responsibility of the applicant, and as it is situated adjacent to Category I buildings, the requested information also applies to this structure:

1. Describe the design process that will be applied to these structures and describe in detail how they will be analyzed for SSE load conditions;

2. For each building (TB, SB, and AB), describe the building design including the eccentrically braced frame in steel structures and the "crumple zone" in concrete structures and how the design prevents seismic interaction with a Category I structure;

3. Describe the collapse sequence and how the collapse will be controlled under an SSE event such that failure occurs in a direction away from a Category I structure; and,

4. Describe how the displacement of the TB and AB will be determined to verify that the separation distance to Category I structures are adequate during an SSE event.

The applicant needs to add the AB to Table 3.7-14 and revise Table 3.2-1 to reflect the fact that the Access Building is now classified by AREVA as a Seismic Category II structure.

Regarding the Circulating Water System (CWS) Makeup Water Intake Structure (MWIS), the applicant in its response states that the embedded portion of the CWS MWIS will be designed as a Seismic Category I structure. Therefore, the design methodology for embedded concrete structure will meet Acceptance Criteria 8.C of SRP 3.7.2. However, it is not clear from the applicant's response whether the operating deck of the CWS MWIS will be designed as a Seismic Category I structure. As failure of this portion of the CWS MWIS could compromise the structural integrity of the embedded walls and the Seismic Category I Common Basemat Intake Structure (CBIS) slab, the applicant is requested to provide a description of the design requirements for the operating deck slab, and if not designed to Seismic Category I requirements, to provide the technical justification and consequences for not doing so.

Also, FSAR Section 3.7.2.3.2 states, in part, that a portion of the pump house enclosure is partially supported on the operating deck slab of the CWS MWIS, and that the masses corresponding to the applicable dead loads and snow loads for the pump house enclosure are appropriately included in the finite element model. The pump house enclosure is supported partially on the operating deck of the CWS MWIS and partially on its own slab. It is not clear from the response if a portion of the dead loads and snow loads associated with the pump house enclosure are included in the CWS MWIS model, or if the entire dead load and snow load is used. Since this can affect the analysis results, the applicant is requested to explain what was done and include this explanation in the FSAR.

Since the pump house enclosure is not designed for seismic loads, the applicant is requested to address what will happen to the operating deck of the CWS MWIS if the enclosure collapses and what will be the effect of this collapse on the seismic response of the CBIS.

If the pump house does not collapse, the response of the CBIS will be influenced by the mass and structural stiffness of the pump house enclosure and the pump house enclosure slab. The applicant is requested to discuss and quantify the effect of the pump house enclosure and slab on the CBIS seismic response and in-structure response spectra (ISRS).

This information will assist in determining whether or not adjacent or near-by Seismic Category I structures will be capable of meeting their intended safety functions under a design basis earthquake event.

Response (Partial, for ESWB and EPGB)

Item 1

The Seismic Category II Turbine Building (TB), Switchgear Building (SB) and Access Building (AB) are located in the vicinity of the Nuclear Island Common Basemat Structures. Using classical finite element analysis methods, these buildings are analyzed and designed to prevent their failure under site-specific SSE loading conditions and to maintain margin of safety equivalent to that of Seismic Category I structures. The structural steel components of these structures are designed using ANSI/AISC N690. The reinforced concrete components of these structures are designed using ACI 349. Therefore, the design methodology for these structures meets NUREG-0800 Section 3.7.2, Acceptance Criterion 8.C.

Item 2

None of the buildings will have "an eccentrically braced frame in steel structures" or "crumple zones in concrete structures." The seismic interaction of the TB, SB and AB with adjacent Seismic Category I structures is prevented by:

- Analyzing and designing to prevent their failure under site-specific SSE loading conditions and maintaining margin of safety equivalent to that of Seismic Category I structures.

- Assuring that the separation distances between these Seismic Category II structures and nearest Seismic Category I structures are greater than the combined elastic deformations.

Item 3

The TB, the SB and the AB are designed to remain elastic under an SSE event. Therefore, none of the buildings is allowed to collapse under an SSE event.

Item 4

The elastic displacements of the TB, the SB and the AB are computed using classical finite element analysis methods. Upon closure of COLA Part 10 Appendix B ITAAC in Tables 2.4-10 (Turbine Building Inspections, Tests, Analyses, and Acceptance Criteria), 2.4-11 (Switchgear Building Inspections, Tests, Analyses, and Acceptance Criteria) and 2.4-32 (Access Building Inspections, Tests, Analyses, and Acceptance Criteria), the finite element analyses report will confirm that the elastic displacements of these buildings combined with those of nearest Seismic Category I structures are less than the provided separation distances.

Paragraph 2

The Access Building is added to the updated FSAR Table 3.7-11 as shown in the attached markup. The FSAR Table 3.2-1 is also updated as shown in the attached markup.

Paragraphs 3, 4, 5 & 6

Response to be provided by May 27, 2011

COLA Impact

The following changes are made to FSAR Table 3.2-1 (Page 1 of 10):

KKS System or Component Code	SSC Description	Safety Classification (Note 1)	Quality Group Classification	Seismic Category (Note 2)	10CFR50 Appendix B Program (Note 5)	Location (Note 3)	Comments/ Commercial Code (Note 10)
~~Table 3.2.2-1 of the U.S. EPR FSAR contains the following conceptual design information for the SM, SN, Cranes, Hoists, and Elevators category for: UKE, Access Building, and UBZ, Buried Conduit Duct Bank.~~							
~~[[UKE~~	~~Access Building~~	~~NS-AQ~~	~~N/A~~	~~II~~	~~No~~	~~UKE~~	
~~UBZ~~	~~Buried Conduit Duct Bank~~	~~S~~	~~N/A~~	~~I~~	~~Yes~~	~~UBZ]]~~	
~~The U.S. EPR FSAR descriptions provided in U.S. EPR FSAR Table 3.2.2-1 regarding the SM, SN, Cranes, Hoists, and Elevators category for: UKE, Access Building, and UBZ, Buried Conduit Duct Bank are applicable to CCNPP Unit 3, and are incorporated by reference.~~							
PED UHS Makeup Water System							

. . . and Page 6 of 10:

Other Site-Specific Structures							
~~UBA~~	~~Switchgear Building~~	~~NS~~	~~E~~	~~CS~~	~~No~~	~~UBA~~	~~IBC/AISC 341/AISC 360/ ACI 318/ACI 349/ANSI/ AISC N690~~
~~UMA~~	~~Turbine Building~~	~~NS~~	~~E~~	~~CS~~	~~No~~	~~UMA~~	~~IBC/AISC 341/AISC 360/ ACI 318/ACI 349/ANSI/ AISC N690~~
UMA, UBA	Turbine Building, Switchgear Building	NS-AQ	NA	II	Yes	UMA, UBA	Steel – AISC N690 Concrete – ACI 349
UKE	Access Building	NS-AQ	NA	II	Yes	UKE	Steel – AISC N690 Concrete – ACI 349
UAC	Grid Systems Control Building	NS	E	CS	No	UAC	IBC
UQZ	Electrical Duct Banks traversing from the Safeguards Buildings to the Four Essential Service Water Buildings and Both Emergency Power Generating Buildings	S	C	I	Yes	UJK/ UZT/ UQB/ UBP	EEE/ACI-349/NEC

The following changes are made to FSAR Section 3.7.2.8

3.7.2.8 Interaction of Non-Seismic Category I Structures with Seismic Category I Systems

. . .

U.S. EPR FSAR Section 3.7.2.8 addresses the interaction of the following Non-Seismic Category I structures with Seismic Category I structures:

- Vent Stack
- Nuclear Auxiliary Building
- Access Building
- Turbine Building
- Radioactive Waste Processing Building

{The following CCNPP Unit 3 Non-Seismic Category I structures identified in Table 3.2-1 could also potentially interact with Seismic Category I SSC:

- Buried and above ground Seismic Category II and Seismic Category II-SSE Fire Protection SSC, including Fire Water Storage Tanks and Fire Protection Building.
- Seismic Category II Turbine Building and Switchgear Building ~~(U.S. EPR FSAR Section 3.7.2.8 also provides conceptual information to address seismic interaction of Turbine Building with the Seismic Category I SSCs)~~
- Seismic Category II Access Building ~~Switchgear Building~~
- Conventional Seismic Grid Systems Control Building
- Seismic Category II Circulating Water Makeup Intake Structure
- Conventional Seismic Sheet Pile Wall.
- Existing Baffle Wall.

The buried Seismic Category II-SSE Fire Protection SSC identified in Table 3.2-1 are seismically analyzed using the design response spectra identified in Section 3.7.1.1.1.4 for use in the analysis of the Seismic Category I site-specific buried utilities. The analysis of the buried Seismic Category II-SSE fire protection SSC will confirm they remain functional during and following an SSE in accordance with NRC Regulatory Guide 1.189 (NRC, 2007). Section 3.7.3.12 further defines the methodology for the analysis of buried Fire Protection piping. Seismic Category II-SSE buried piping is an embedded commodity that by its nature does not significantly interact with above ground Seismic Category I SSC. The buried Seismic Category II-SSE Fire Protection SSCs are designed to the same requirements as the buried Seismic Category I SSCs.

The above ground Seismic Category II and Seismic Category II□SSE Fire Protection SSC, including Fire Water Storage Tanks and Fire Protection Building, identified in Table 3.2-1 are seismically analyzed utilizing the appropriate design response spectra. Seismic load combinations are developed in accordance with the requirements of ASCE 43-05 (ASCE, 2005) using a limiting acceptance condition for the structure characterized as essentially elastic behavior with no damage (i.e., Limit State D) as specified in the Standard. The analysis of the above ground Seismic Category II-SSE fire protection SSC will confirm they remain functional during and following an SSE in accordance with NRC Regulatory Guide 1.189 (NRC, 2007). The analysis of the above ground Seismic Category II fire protection SSCs will confirm they maintain a pressure boundary after an SSE event.

Table 3.7-11 provides the criteria used to prevent seismic interaction of Turbine Building, Switchgear Building, Access Building, Circulating Water Makeup Intake Structure and Grid Systems Control Building with other Seismic Category I structures, systems and components (SSCs).

The Seismic Category II Turbine Building (TB), Switchgear Building (SB) and Access Building (AB) are located in the vicinity of the Nuclear Island Common Basemat Structures. These buildings are analyzed and designed to prevent their failure under site-specific SSE loading conditions and to maintain margin of safety equivalent to that of Seismic Category I structures. The structural steel components of these structures are designed using ANSI/AISC N690 (ANSI/AISC, 2004). The reinforced concrete components of these structures are designed using ACI 349 (ACI, 2001). Therefore, the design methodology for these structures meets NUREG-0800 Section 3.7.2, Acceptance Criterion 8.C (NRC, 2007a). During detailed design, the elastic displacements of the TB, the SB and the AB will be computed using classical finite element analysis methods. The elastic displacements will be combined with those of the nearest Seismic Category I structures. It will be confirmed that the combined elastic displacements are less than the provided separation distances. ~~The Seismic Category II Turbine Building and Seismic Category II Switchgear Building together comprise a common Turbine Island (TI) structure and are situated approximately 30 ft (9.1 m) from the NI Common Basemat structures. The Switchgear Building is a steel framed structure. The Turbine Building and Switchgear Building are designed using conventional seismic codes and standards presented in Table 3.7-11, but are also analyzed and designed using Site SSE to prevent seismic interaction with the Seismic Category I SSCs. An evaluation of the site-specific SSE responses will confirm that the separation distance between the TI structure and the Seismic Category I SSCs exceeds the sum of the maximum relative seismic displacement between the structures, construction tolerances and settlement effects by an appropriate factor of safety.~~

The Conventional Seismic Grid Systems Control Building is located in the Switchyard area, and has a minimum separation distance of approximately 700 ft (213.4 m) from the nearest Seismic Category I SSCs (see Figure 2.1-5). Therefore, potential collapse of this building has no adverse impact on the function of Seismic Category I SSCs. This meets NUREG-0800 Section 3.7.2, Acceptance Criterion 8.A (NRC, 2007a).

The following changes are made to section 3.7.2.16

3.7.2.16 References

{**ACI, 2006.** Seismic Design of Liquid-Containing Concrete Structures, ACI 350.3-06, American Concrete Institute, 2006.

ACI, 2001. Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety-Related Concrete Structures, ACI 349-01/349-R01, American Concrete Institute, 2001.

ANSI/AISC, 2004. Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, 1994 including Supplement 2, ANSI/AISC N690,American National Standards Institute, 2004.

ASCE, 2000. Seismic Analysis of Safety-Related Nuclear Structures and Commentary, ASCE Standard 4-98, American Society of Civil Engineers, 2000.

The following changes are made to Table 3.7-11

Table 3.7-11— {Criteria for Seismic Interaction of Site-Specific Non-Seismic Category I Structures with Seismic Category I Structures}

Basis: Control Interaction through Prevention of Structure-to-Structure Impact[1]				
Structure	**Seismic Category**	**Design Code**	**Seismic Interaction Criteria**	**Seismic Interaction Evaluation**
Turbine Building and Switchgear Building	SC-II[2b]	~~IBC~~ Steel – ~~AISC 341, AISC 360 &~~ AISC N690[3] Concrete – ~~ACI 318 &~~ ACI 349[3]	SSE	No Interaction
Grid Systems Control Building	CS[2a]	IBC Steel – AISC 360 Concrete – ACI 318	None	No Interaction
Circulating Water Intake Structure	SC-II[2b]	IBC Steel – AISC 341 & ACI 360 Concrete – ACI 349	SSE	No Interaction
Access Building	SC-II[2b]	Steel – AISC N690[3] Concrete – ACI 349[3]	SSE	No Interaction

Notes:
1. This table is not applicable to equipment and subsystems qualification criteria.
2. Seismic Classification
   a. Conventional Seismic
   b. Seismic Category II
3. AISC N690 and ACI 349, as applicable, will be used for SSE and tornado load combinations ~~in the design of the Lateral Force Resisting System (LFRS)~~.

RAI 253

Question 03.07.02-49

The applicant is requested to provide the following additional information regarding the determination of factors of safety against overturning and sliding:

The applicant stated that the results of the SASSI analysis are used for the overturning and sliding calculations of the EPGB, ESWB, CBIS, and UHS EB. The applicant is requested to describe whether or not during the seismic response of these structures there exists a vertical tensile force between the bottom of the basemat and the supporting subgrade. Since this would indicate an uplift of the mat where such a force is indicated, the applicant is requested to describe the effect this has on the results of the overturning and sliding analysis and whether the results of these analyses can be considered as valid if such an effect exists.

The load combination for determining the factor of safety against overturning and sliding as provided in RG 3.8.5, Acceptance Criteria 3 is the sum of the dead load, lateral earth pressure loads and the seismic load. FSAR Table 3E.4-1 agrees with the SRP load combination. However, in FSAR Section 3.7.2.14, it states that in addition to the self weight of the structure, weight of the permanent equipment and contained water during normal operation, 25 percent of the design live load and 75 percent of the design snow load is also included to determine the restoring moments. The applicant is requested to explain this inconsistency between the different sections of the FSAR, and provide justification for including these loads for determination of sliding and overturning factors of safety.

It is not clear how static and dynamic lateral earth pressures were determined in the analysis and how they contribute to the overturning and sliding factors of safety for the structures. The applicant is requested to provide a description of how lateral earth pressures were determined and if tensile soil forces are indicated between the building sidewalls and the soil to describe how these were treated in the overturning and sliding analyses.

The staff requests this information to assist in understanding the applicant's methodology and to enable the staff to conclude whether or not the structure meets the minimum factors of safety for sliding and overturning stability. The applicant is requested to include in its response an appropriate update to the FSAR.

Response

This response is divided into two parts. Part one addresses the EPGB and ESWB. Part two addresses the CBIS and UHS Electrical Building.

Part 1, EPGB and ESWB

<u>Vertical Tensile Stresses</u>

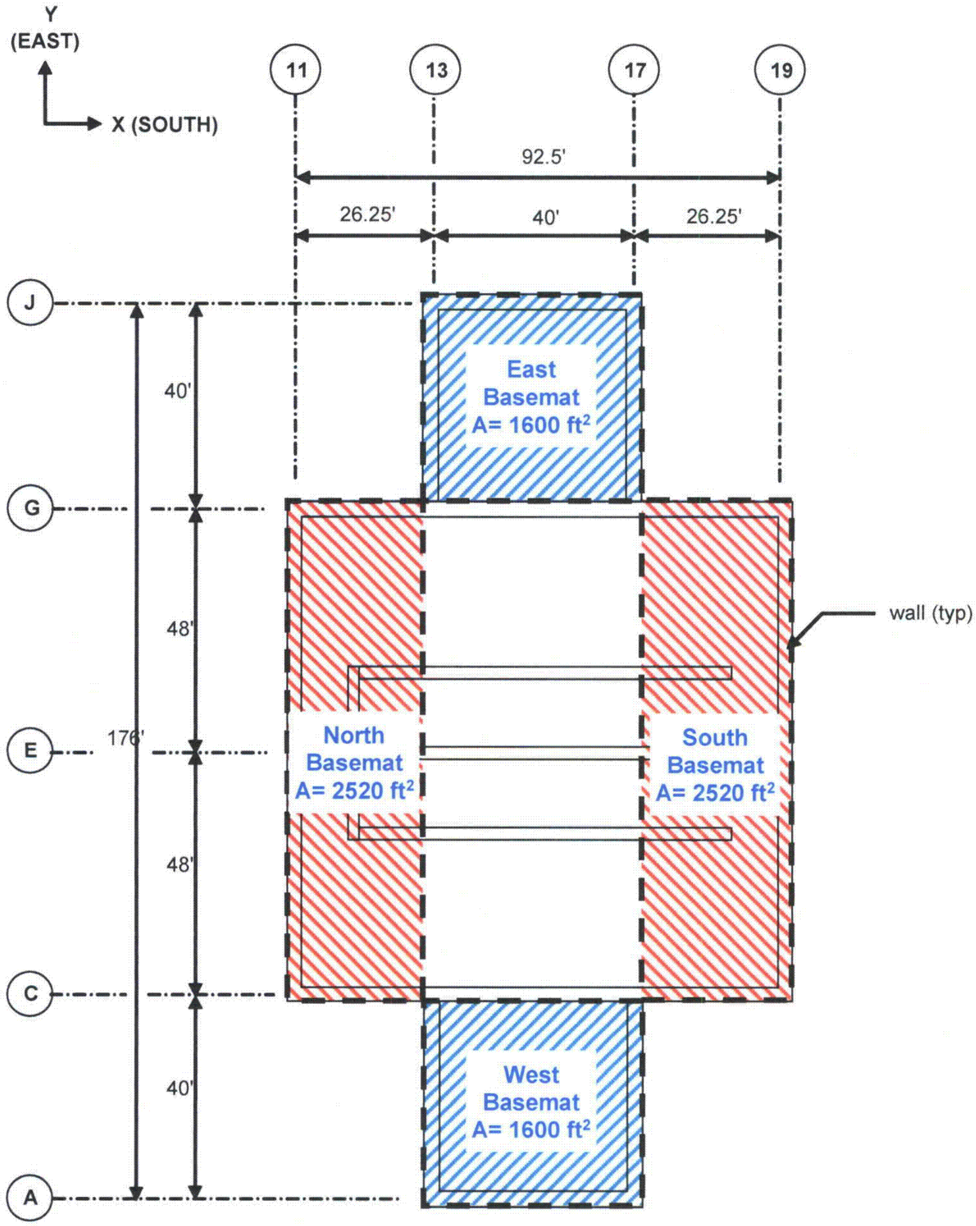
Finite element SSI models of the EPGB and ESWB have stiff springs connecting each node of their basemats to the supporting soil nodes. Forces in these springs represent reaction forces from the supporting soil to the basemat. To address dynamic side soil pressure, stiff springs are also added to the embedded exterior walls of the ESWB connected wall nodes with the side soil nodes. As demonstrated in the SSI calculations for the EPGB and ESWB, maximum accelerations and maximum spring forces are from the Upper Bound soil case. Therefore, in addressing soil separation and uplift, only upper bound soil case is considered.

For both the EPGB and the ESWB, net vertical seismic spring forces at 4 zones (east, west, north, and south) of the basemat, due to seismic input motions in 3 global directions, are calculated for the controlling upper bound soil case and compared with the tributary weight of each zone. The four zones of the EPGB basemat and ESWB basemat are shown in Figure 1 and Figure 2, respectively. The net spring force (time history) at each zone is calculated by (1) algebraic summation of each vertical spring force time history due to seismic input in NS, EW, and Vertical directions, and then (2) algebraic summation of the above combined spring force time histories of all of the nodes within each zone.

These net spring force time histories for the various zones are plotted with the tributary weight (labeled as uplift capacity) of each zone in Figure 3 through Figure 6 for the EPGB and in Figure 7 through Figure 10 for the ESWB. The comparisons in these figures show that the total vertical seismic spring forces within each zone are less than the tributary weight for each zone. Therefore, there is no separation between the basemat and the soil below it, there is no uplift for these two SSI models, and there is no impact to the sliding and overturning evaluations in the calculations.

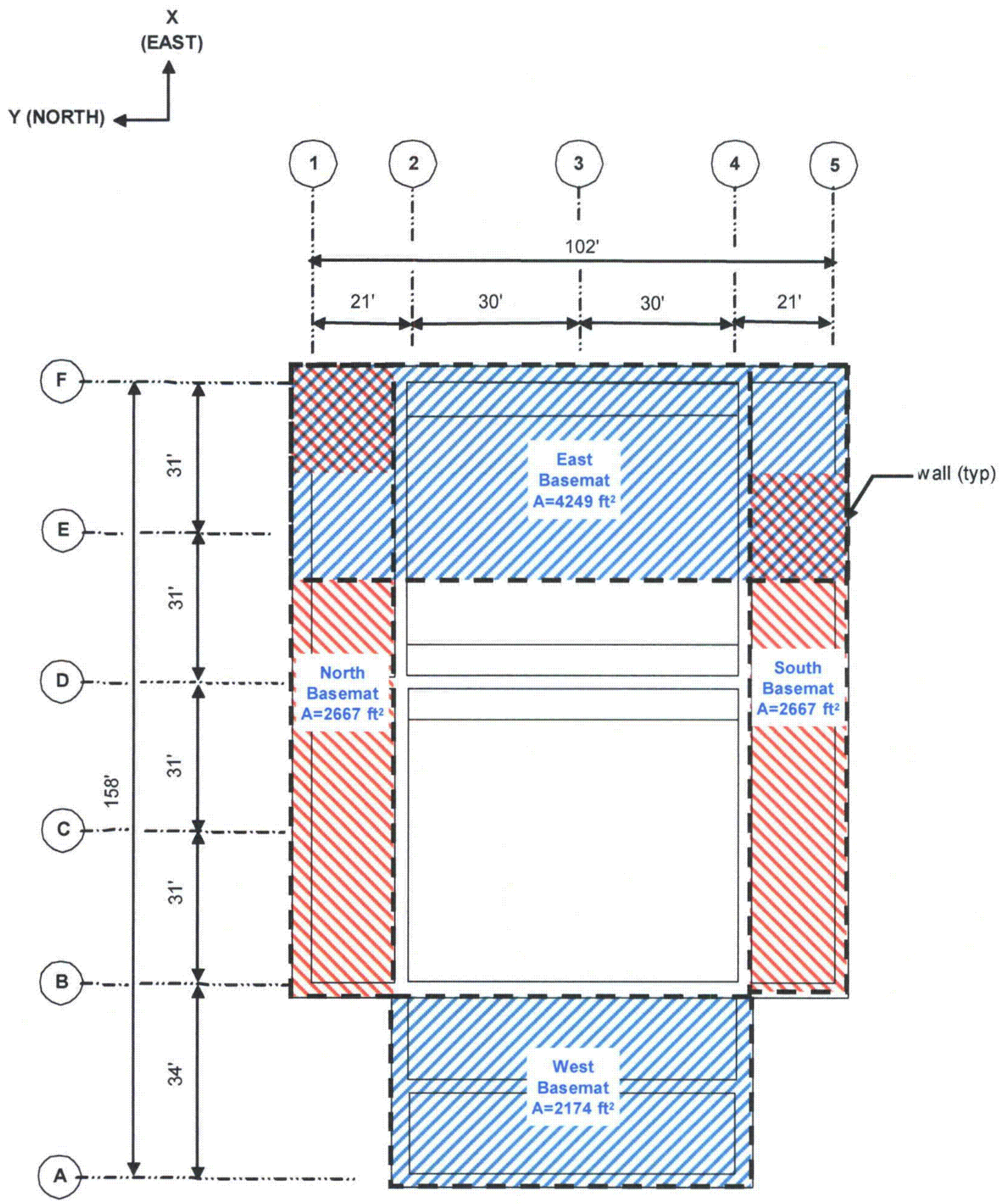
There are two ESWB SSI models, the high water level (HWL) and the low water level (LWL) models where the water stored inside the building structure is at the maximum and minimum levels. For the above evaluation, HWL is conservatively used to calculate the seismic spring forces since that will produce maximum seismic demand, and LWL is conservatively used to calculate the uplift capacities (which is the static weight of the building) since that will produce minimum capacity.

Figure 1. Plan View – Basemat of EPGB



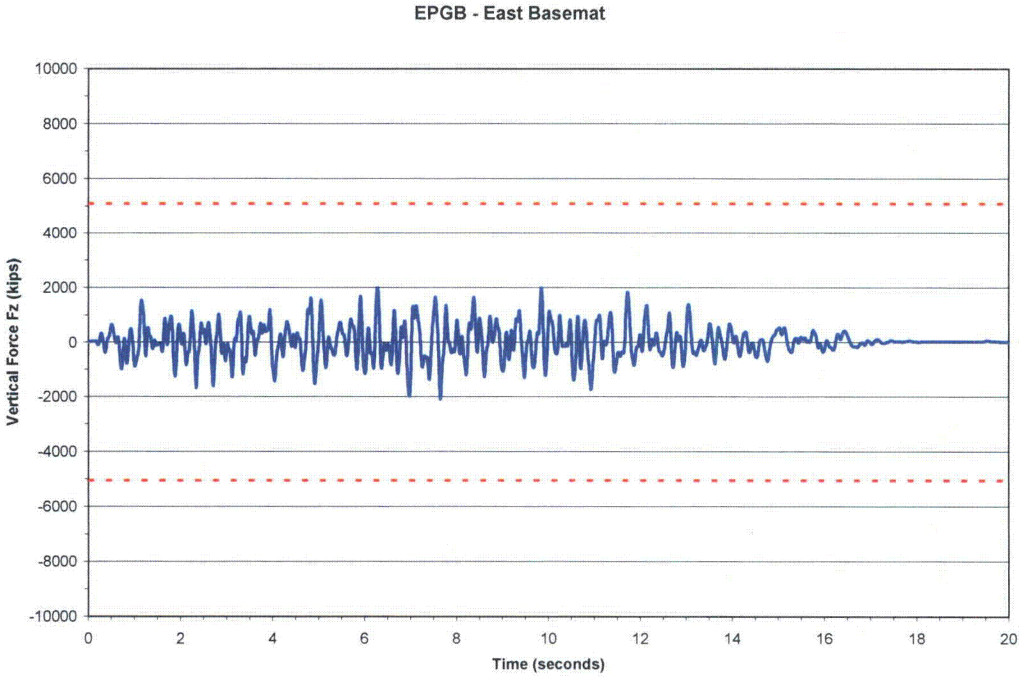
Total of Basemat Area (A) of EPGB Model= 12080 ft^2

Figure 2. Plan View – Basemat of ESWB



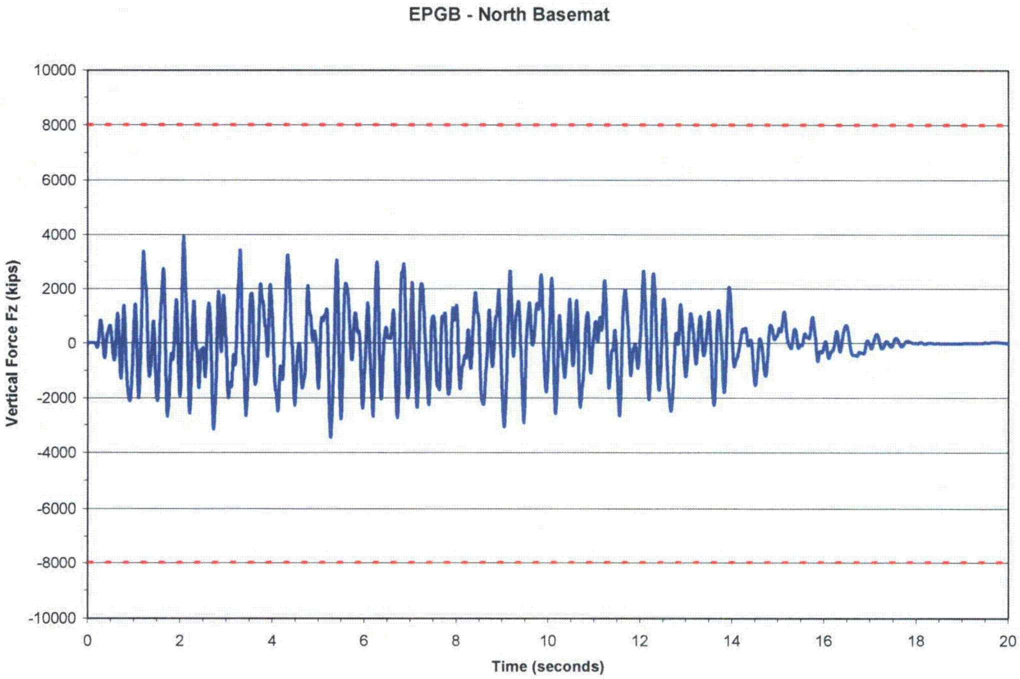
Total of Basemat Area (A) of ESWB Model= 15509 ft^2

Figure 3. Uplift Force under East Basemat, EPGB



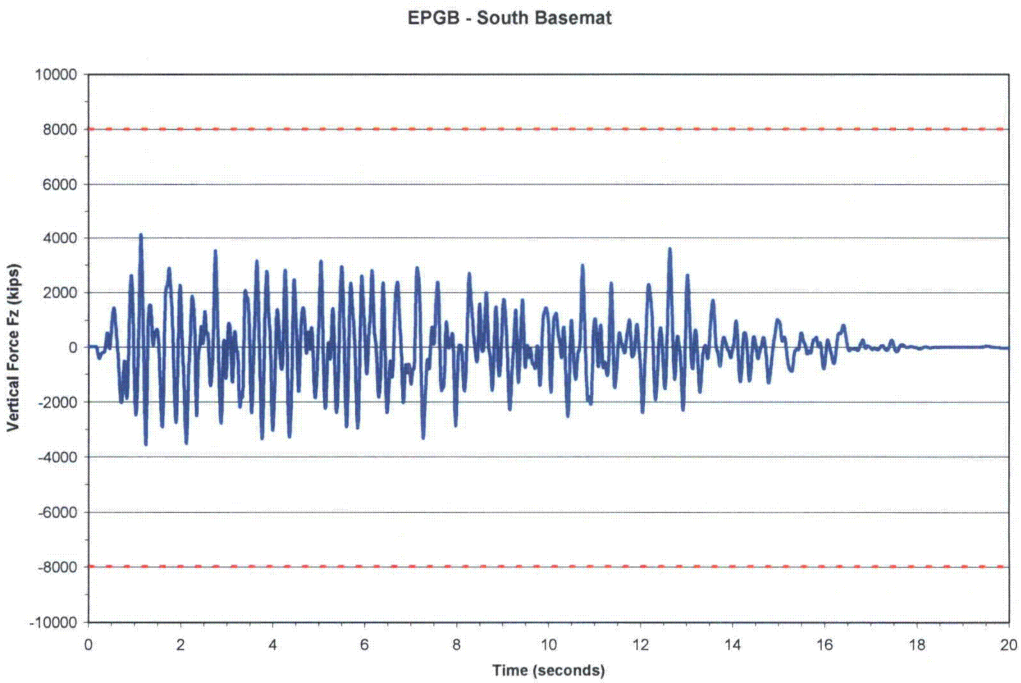
(Dashed red lines are capacity against uplift.)

Figure 4. Uplift Force under North Basemat, EPGB



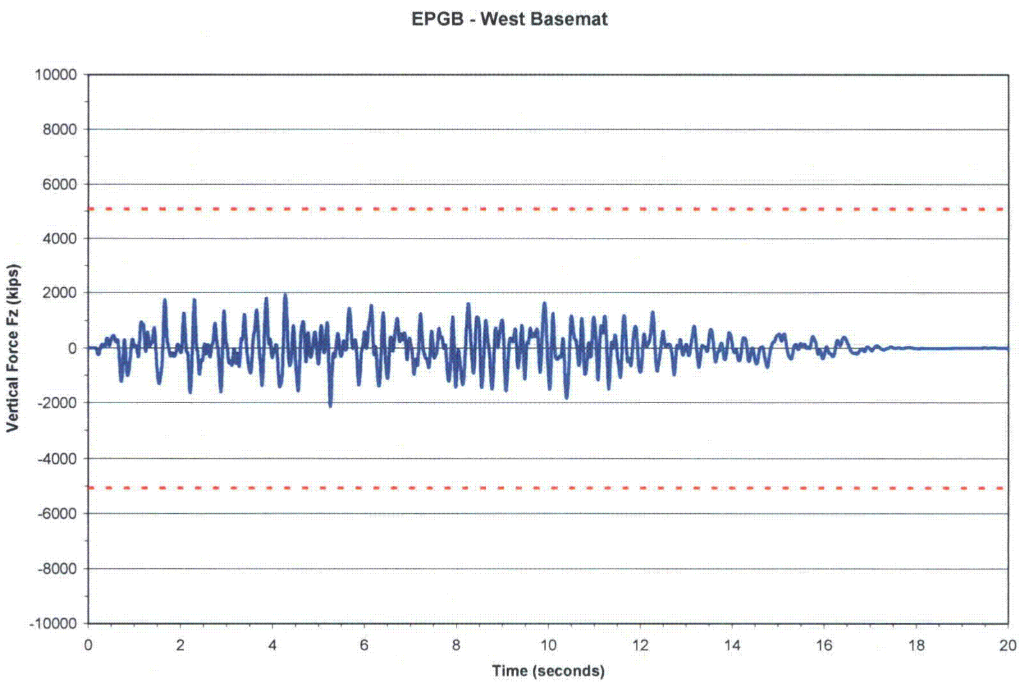
(Dashed red lines are capacity against uplift.)

Figure 5. Uplift Force under South Basemat, EPGB



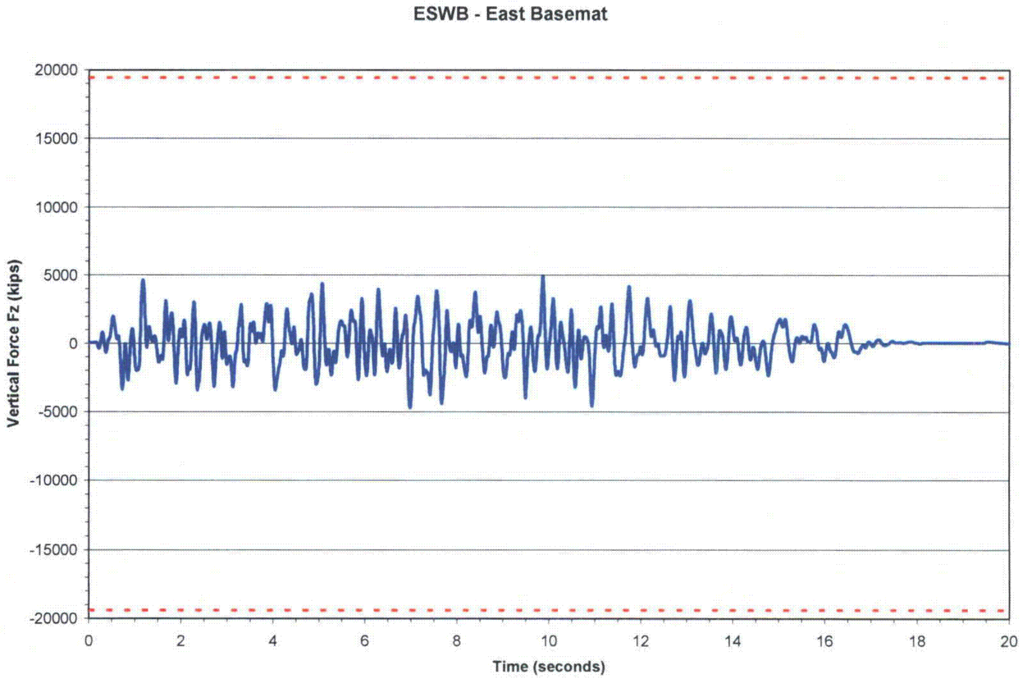
(Dashed red lines are capacity against uplift.)

Figure 6. Uplift Force under West Basemat, EPGB



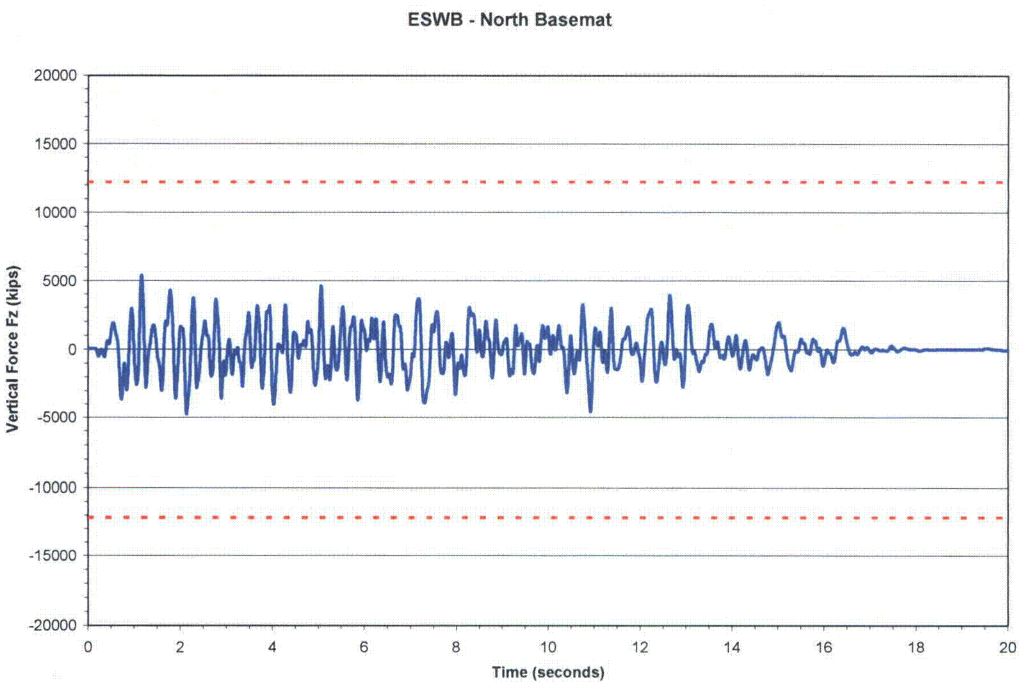
(Dashed red lines are capacity against uplift.)

Figure 7. Uplift Force under East Basemat, ESWB



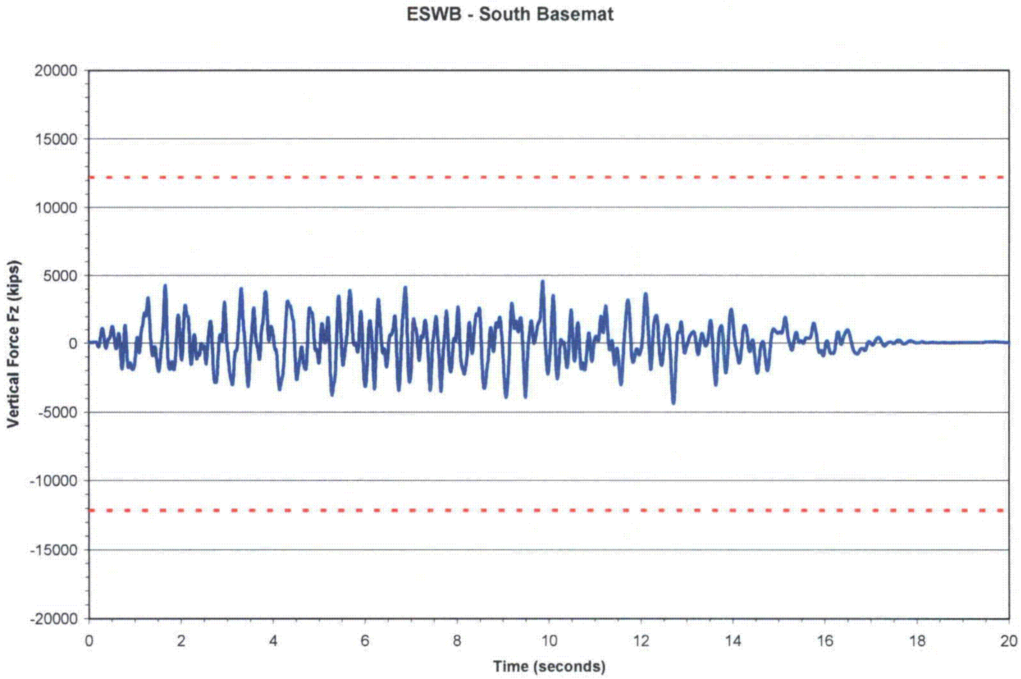
(Dashed red lines are capacity against uplift.)

Figure 8. Uplift Force under North Basemat, ESWB



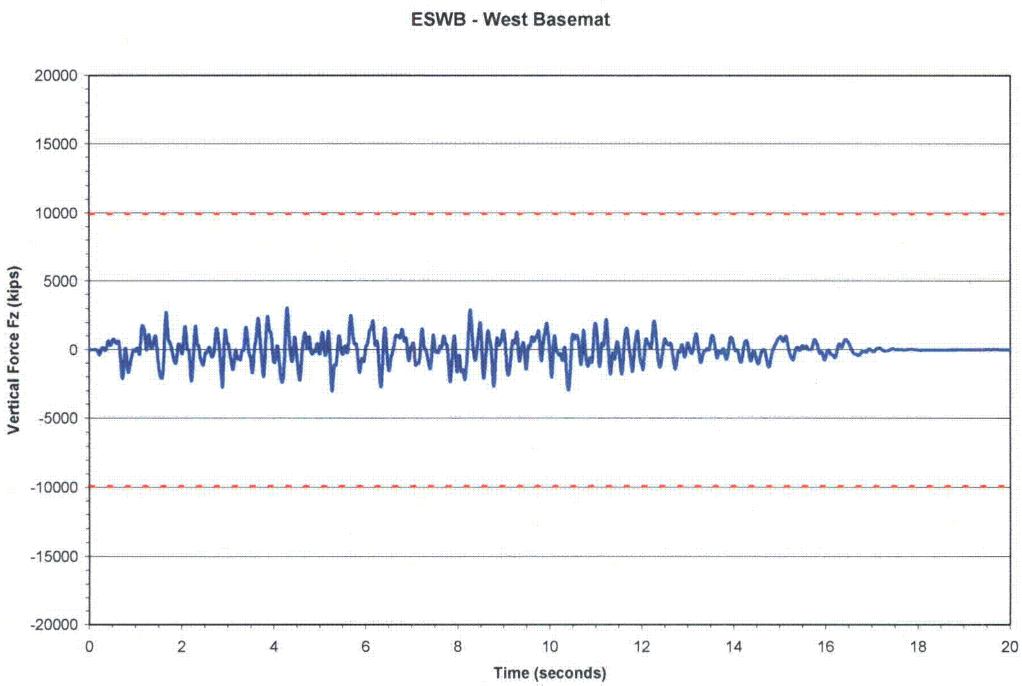
(Dashed red lines are capacity against uplift.)

Figure 9. Uplift Force under South Basemat, ESWB



(Dashed red lines are capacity against uplift.)

Figure 10. Uplift Force under West Basemat, ESWB



(Dashed red lines are capacity against uplift.)

Effect of 25 % of the Design Live Load and 75 % of the Design Snow Load

UNE Letter UN#10-193[1] revised COLA FSAR Section 3.8 and Appendix 3E. This information has been incorporated into COLA FSAR Revision 7. Revised COLA FSAR Subsection 3.8.5.3 provides the stability load combinations, which are consistent with NUREG-0800 Standard Review Plan 3.8.5. COLA FSAR Subsection 3.7.2.14.2 states "The stability of EPGB, ESWB....is determined using the stability load combinations provided in NUREG-0800 Section 3.8.5, Acceptance Criteria 3", and this is consistent with the revised COLA FSAR Section 3.8.5. The description of stability load combinations and associated evaluations has been deleted from the revised COLA FSAR Appendix 3E, which now refers back to COLA FSAR Subsection 3.8.5.3 for the stability evaluation. Therefore, there is no inconsistency between the mentioned COLA FSAR Sections.

The determination of restoring moments includes the effect of 25% live load and 75% snow load since these loads are included in the SSI model that is used to calculate the seismic demands, in accordance with the requirements of NUREG-0800 Section 3.7.2, Acceptance Criteria 3D. The stability evaluation would be overly conservative if these loads are included in calculating the seismic demands but are excluded in calculating the restoring capacities. Additionally, it is noted that 25% live load and 75% snow load represent a small fraction (less than 4%) as compared with the dead loads and weight of the contained water (wherever applicable), and thus the inclusion of 25% live load and 75% snow load has an insignificant effect on the stability evaluation.

Lateral Earth Pressures

The sliding and overturning evaluation that is documented in the SSI calculation considered demand and capacity from the basemat only, while effects from the side wall and side soil are neglected. The omission of side soil effect on the sliding and overturning evaluation is conservative. However, the static and dynamic earth pressures on the side wall are evaluated in the new soil separation calculation as described above to show that soil separation is not a concern and will not affect the SSI analysis results.

To determine if there is separation between embedded exterior side walls and the surrounding soil, total spring forces of a vertical strip of the exterior wall (in the normal to the exterior wall direction) from the SSI analysis of the controlling upper-bound soil case are compared to the static lateral soil pressure. These total spring forces are calculated using the same approach as those used in the above summation of net vertical spring forces below the basemat. It is noted that this evaluation is performed for the ESWB only since the EPGB is a surface founded structure without embedded walls. The static lateral soil pressure is obtained by estimating compaction-induced earth pressures from the grade down to a depth that equates the at-rest soil pressure and then by following the at-rest soil pressure to the embedment depth of the embedded walls. No water pressure is applied since the post construction ground water level is approximately 30 ft below grade, which is below the basemat of the ESWB.

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[1] G. Gibson (UniStar Nuclear Energy) to Document Control Desk ((NRC), "Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 144, Other Seismic Category I Structures, and RAI No. 145, Foundations," letter UN#10-193 dated July 23, 2010.

The comparisons at various selected locations of the ESWB (see Figure 11) are shown in Figure 12 through Figure 29. In these figures, solid red lines are static compressive soil pressure (force) for the tributary width of the sidewalls, based on the static soil pressure used for structural design of the embedded walls; and dotted red lines are a lower bound estimation of the static soil pressure (force) for reference. It is noted that Figure 17 and Figure 18 show comparison at the same location (see Figure 11), with Figure 18 (North 3 A) representing a wider tributary width of the basement wall. Similarly, Figure 23 represents a wider section of the south basement wall than Figure 24.

These comparisons show that there is only a limited number of exceedance where the tension forces between the wall and the soil are higher than the static lateral soil pressure (force). These exceedances are highly localized and very short in duration. Therefore the effects of soil separation at the basement walls on the results of the ESWB SSI analyses are concluded to be negligible and the embedded exterior walls are considered to be always in contact with the side soil.

Figure 11. Plan View – Basement Wall Locations - ESWB

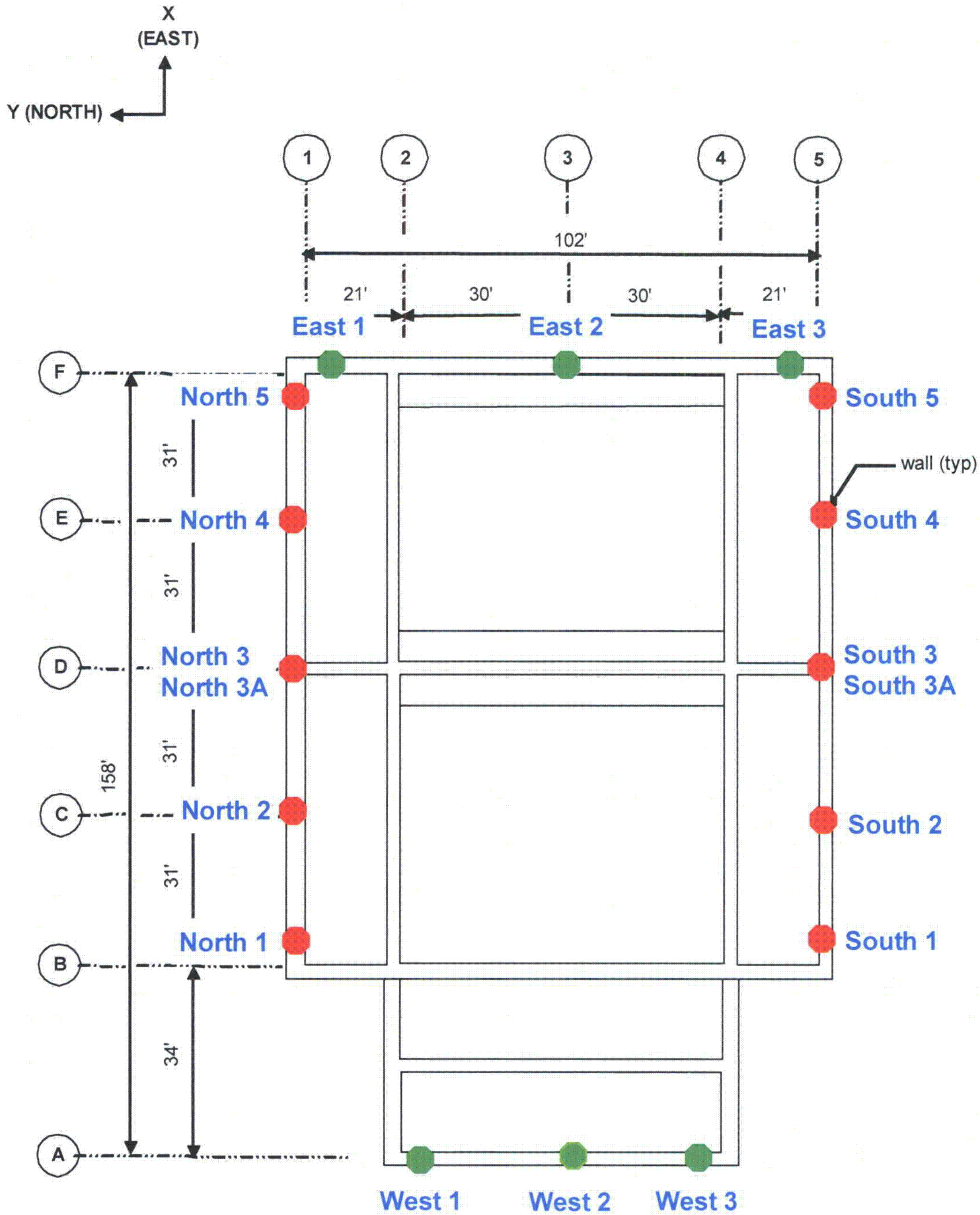
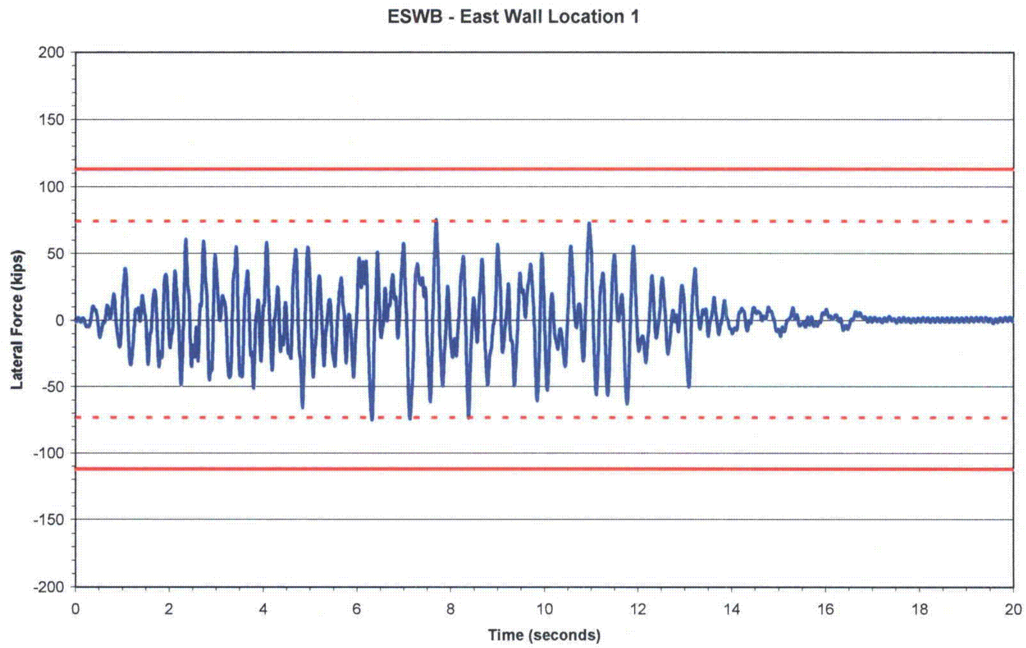
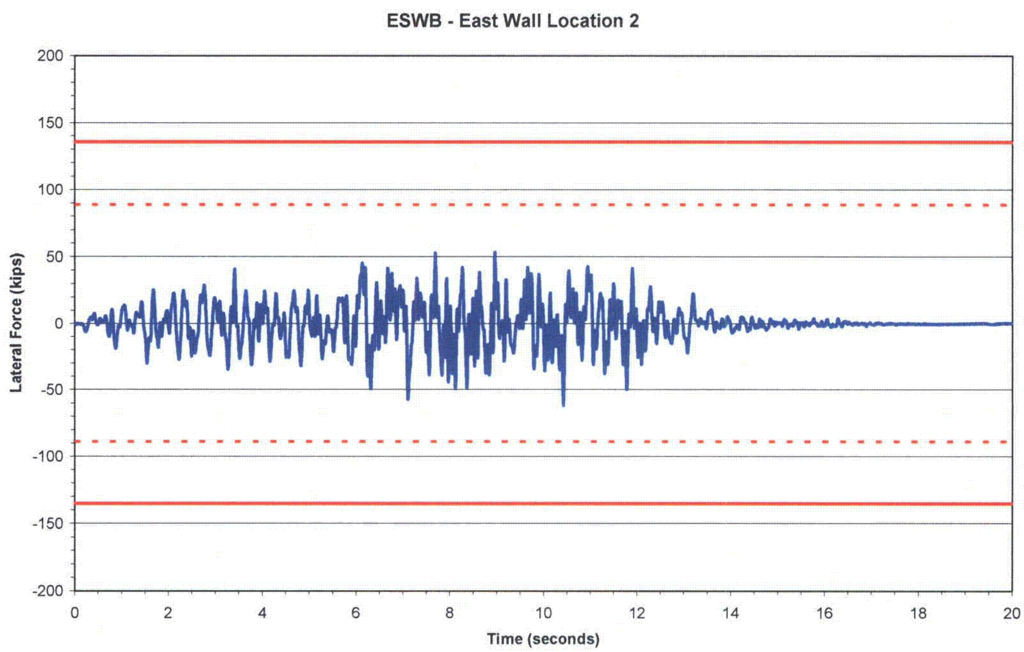


Figure 12. Lateral Force at East Basement Wall Location 1, ESWB



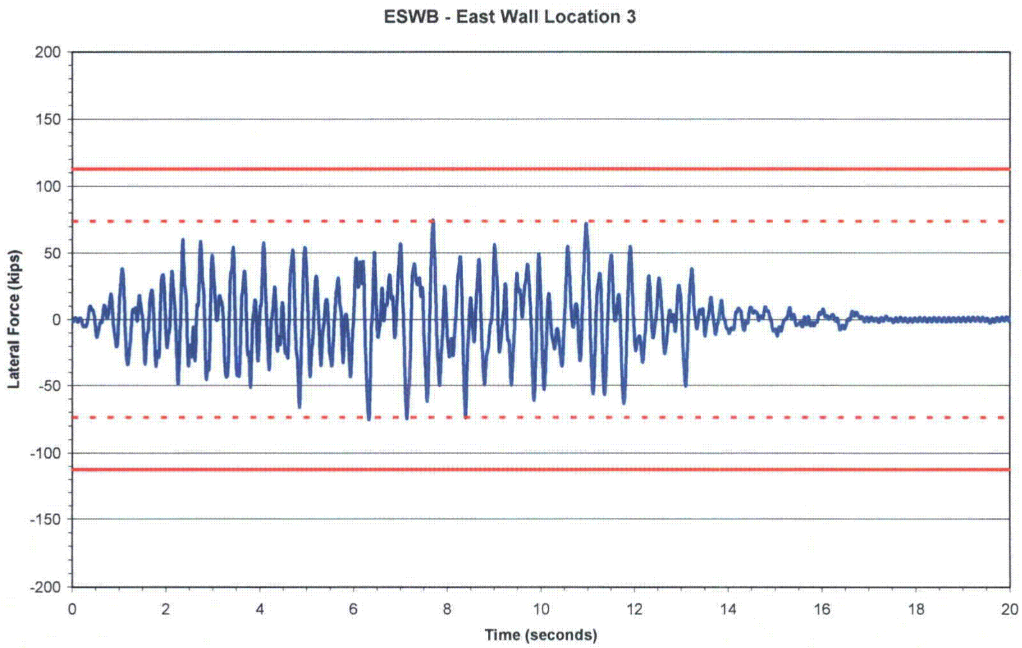
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 13. Lateral Force at East Basement Wall Location 2, ESWB



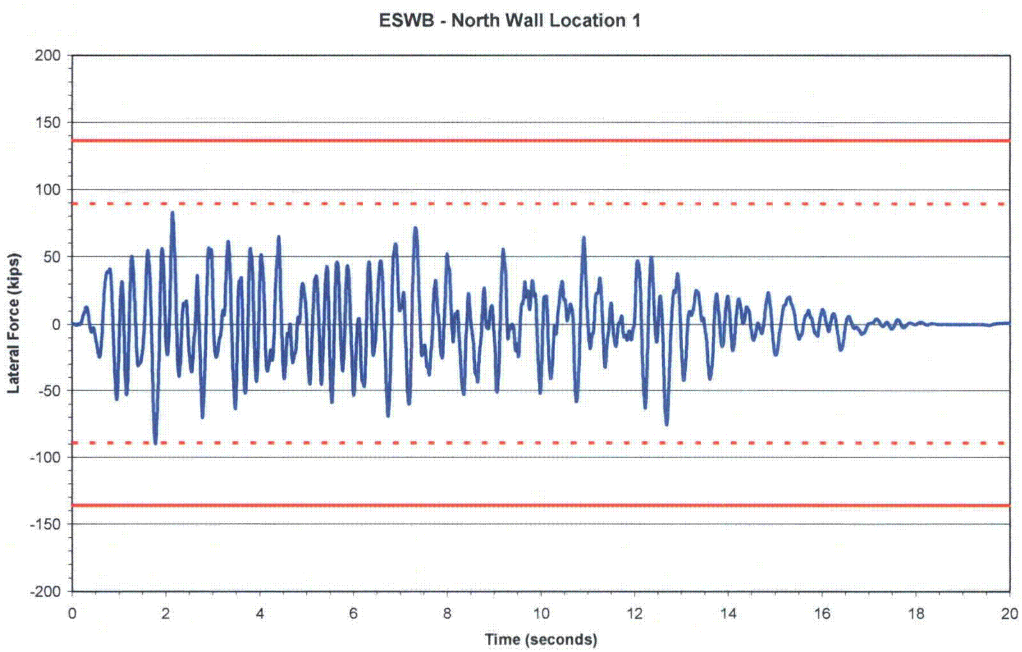
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 14. Lateral Force at East Basement Wall Location 3, ESWB



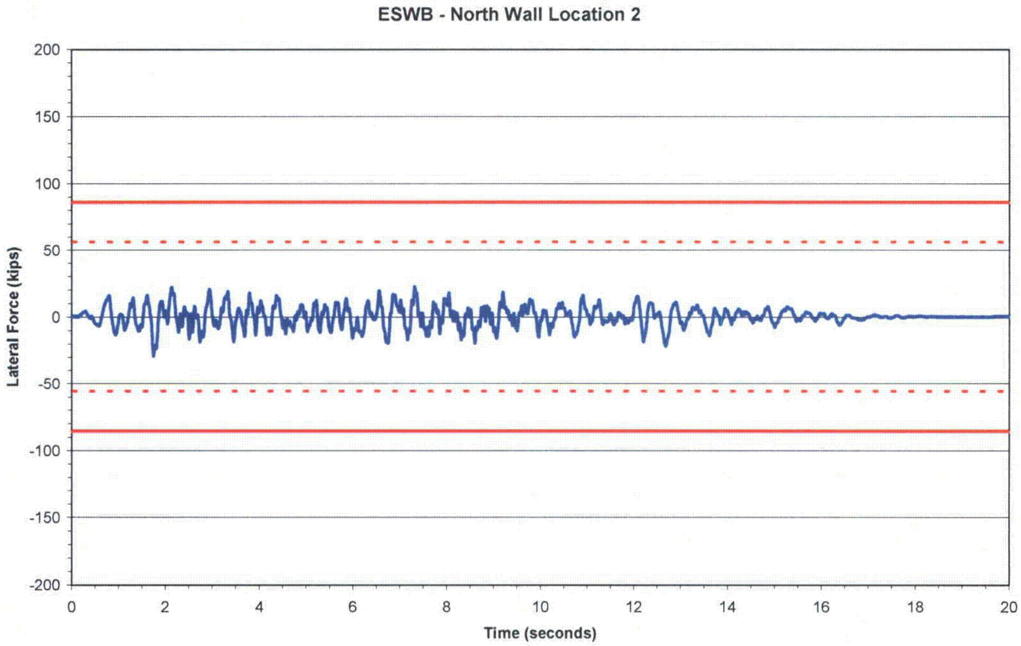
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 15. Lateral Force at North Basement Wall Location 1, ESWB



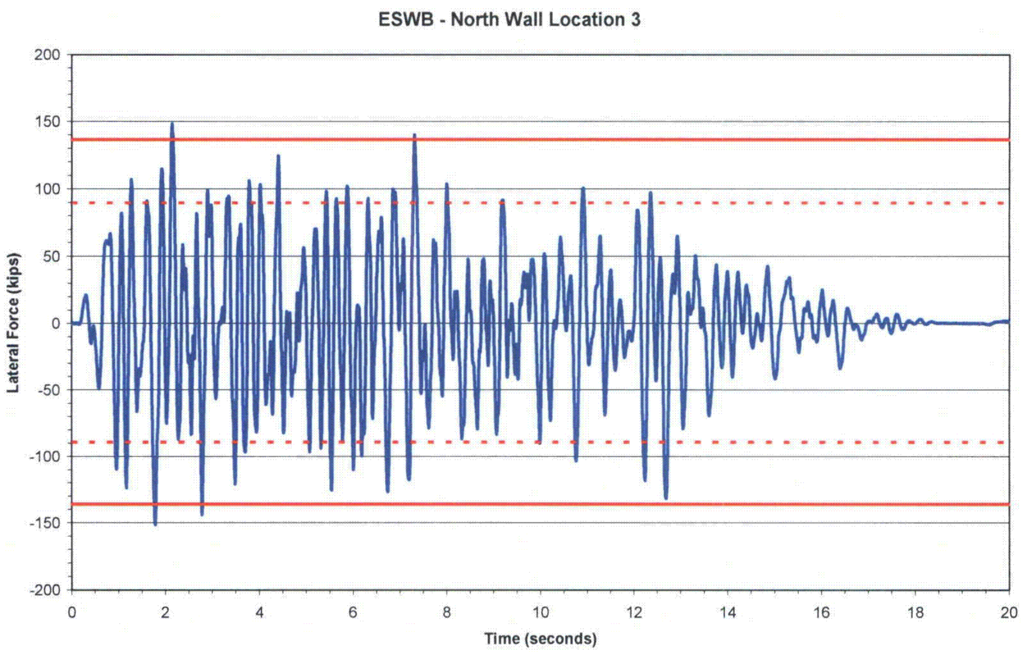
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 16. Lateral Force at North Basement Wall Location 2, ESWB



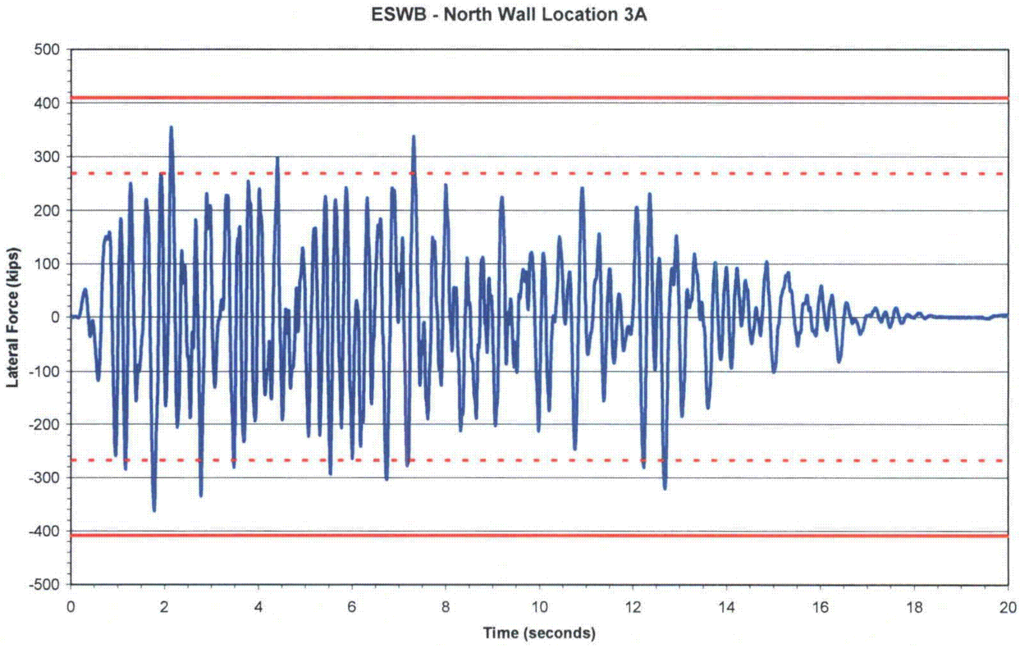
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 17. Lateral Force at North Basement Wall Location 3, ESWB



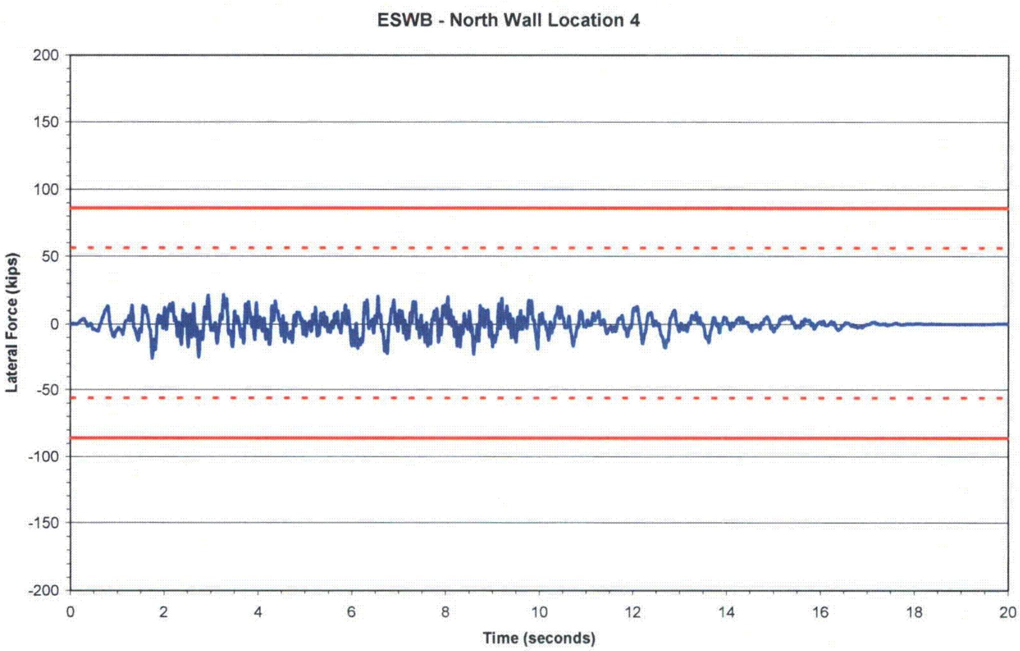
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 18. Lateral Force at North Basement Wall Location 3A, ESWB



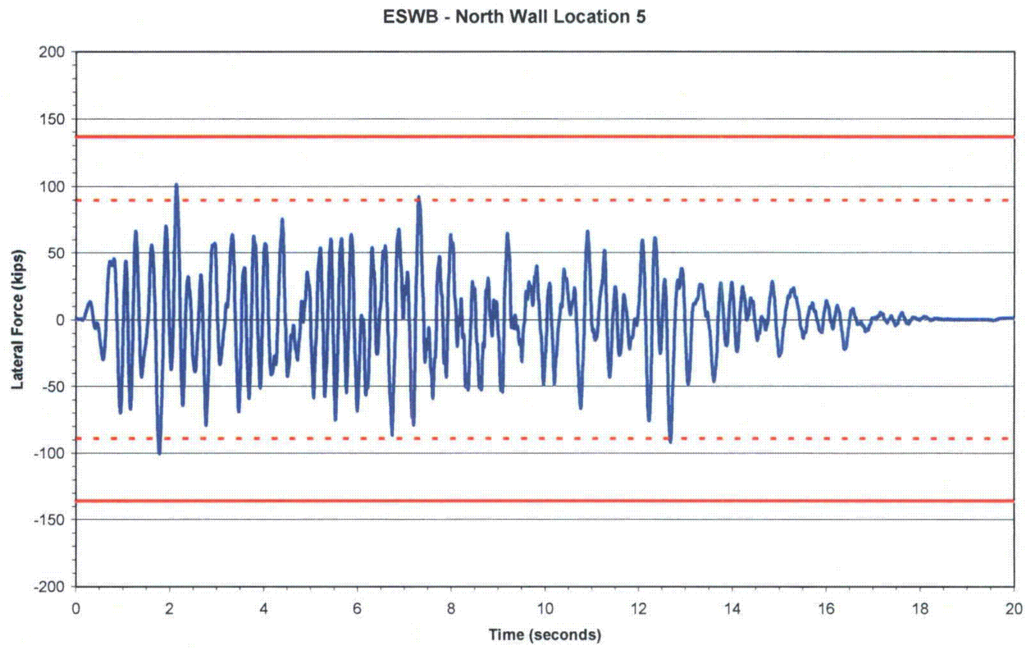
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 19. Lateral Force at North Basement Wall Location 4, ESWB



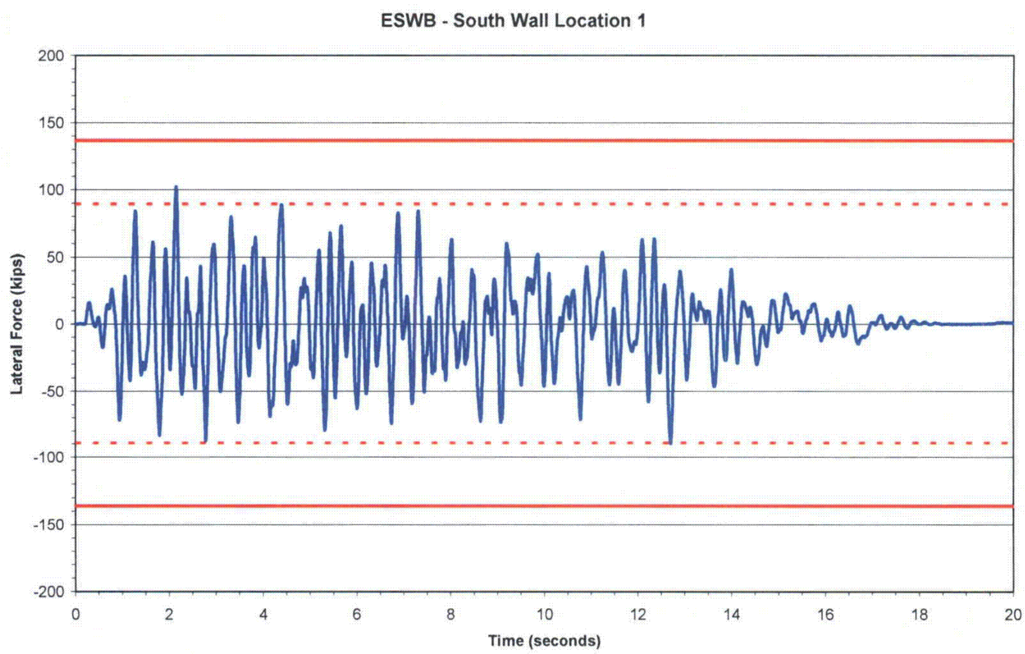
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 20. Lateral Force at North Basement Wall Location 5, ESWB



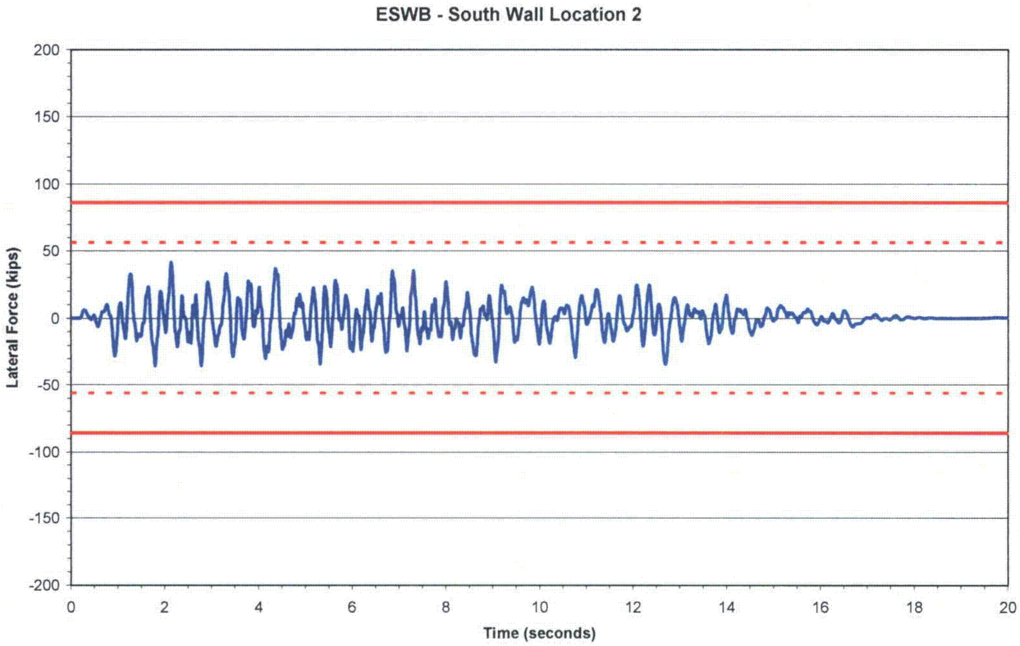
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 21. Lateral Force at South Basement Wall Location 1, ESWB



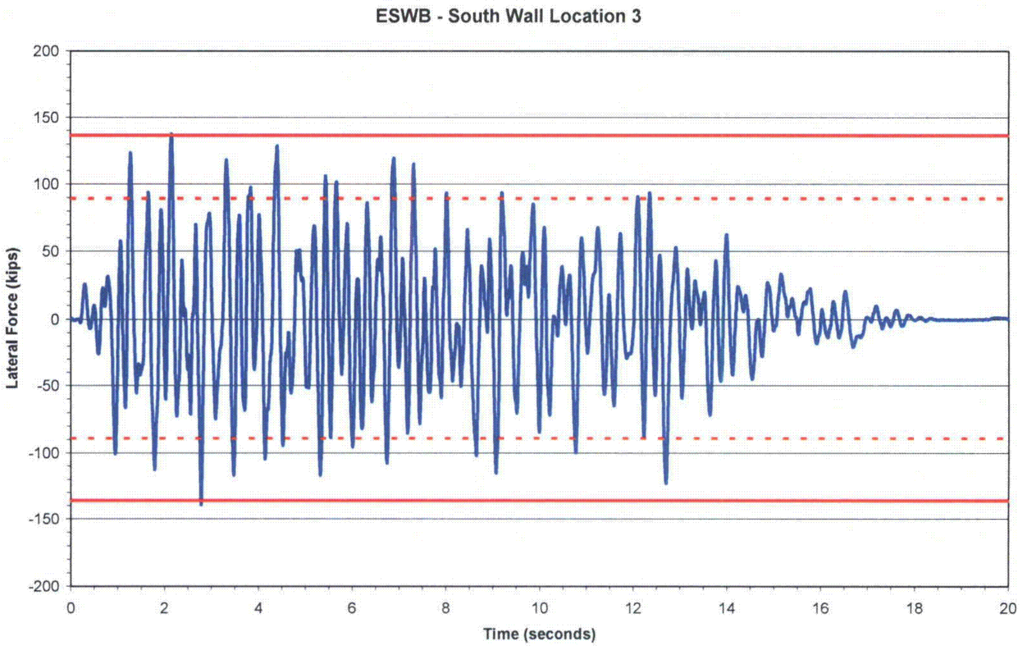
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 22. Lateral Force at South Basement Wall Location 2, ESWB



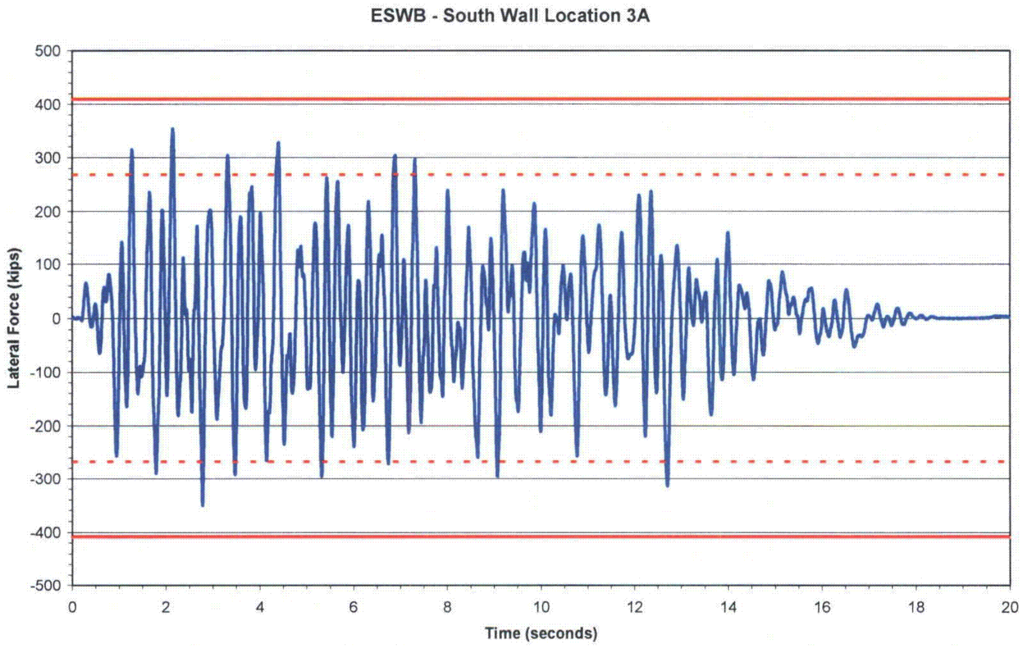
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 23. Lateral Force at South Basement Wall Location 3, ESWB



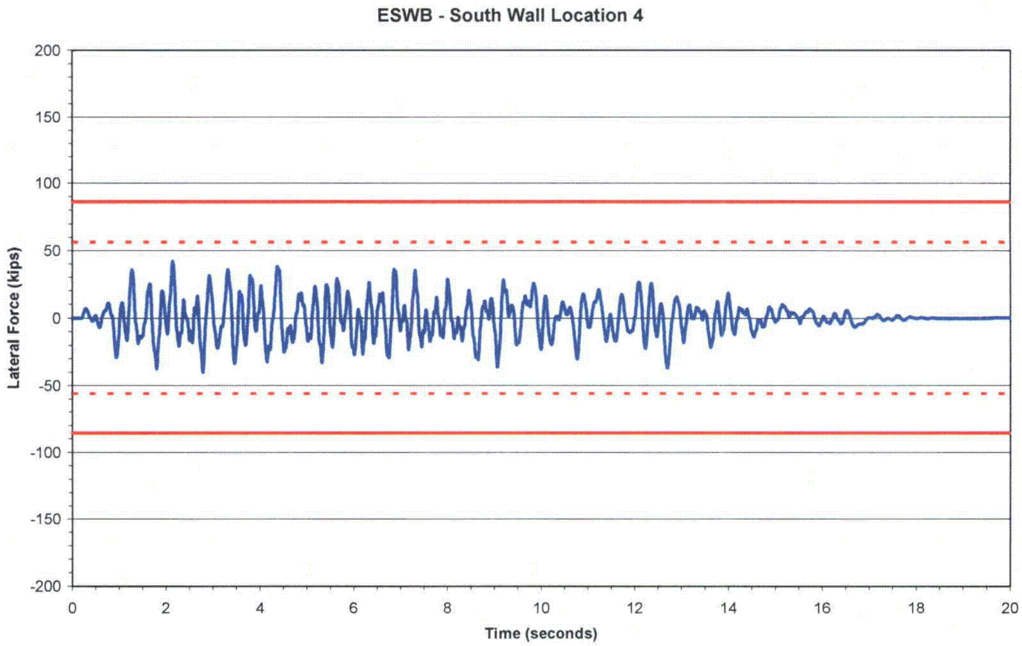
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 24. Lateral Force at South Basement Wall Location 3A, ESWB



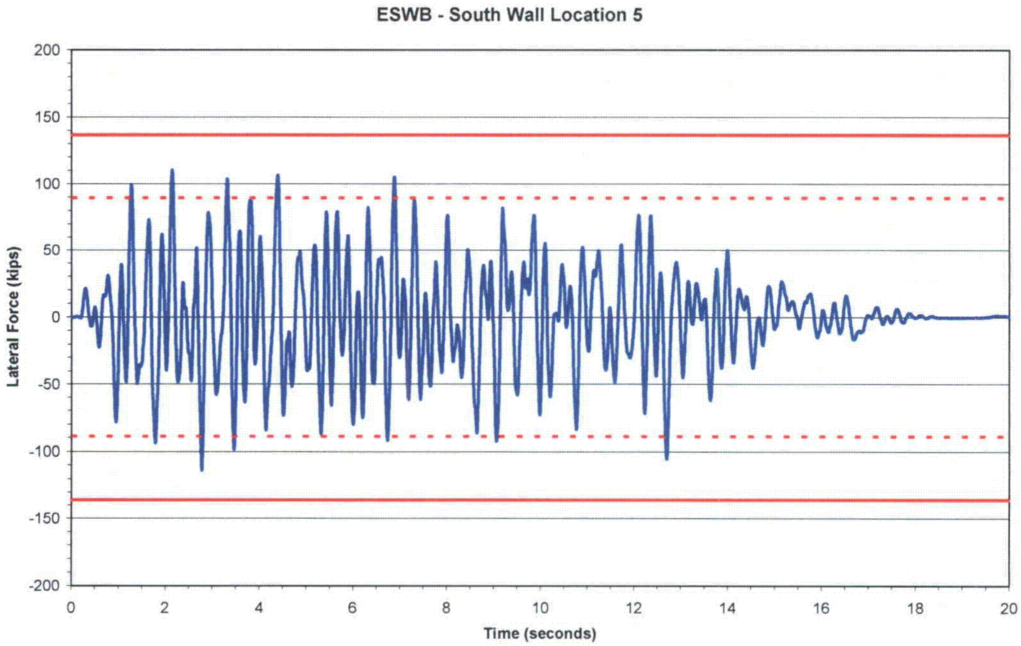
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 25. Lateral Force at South Basement Wall Location 4, ESWB



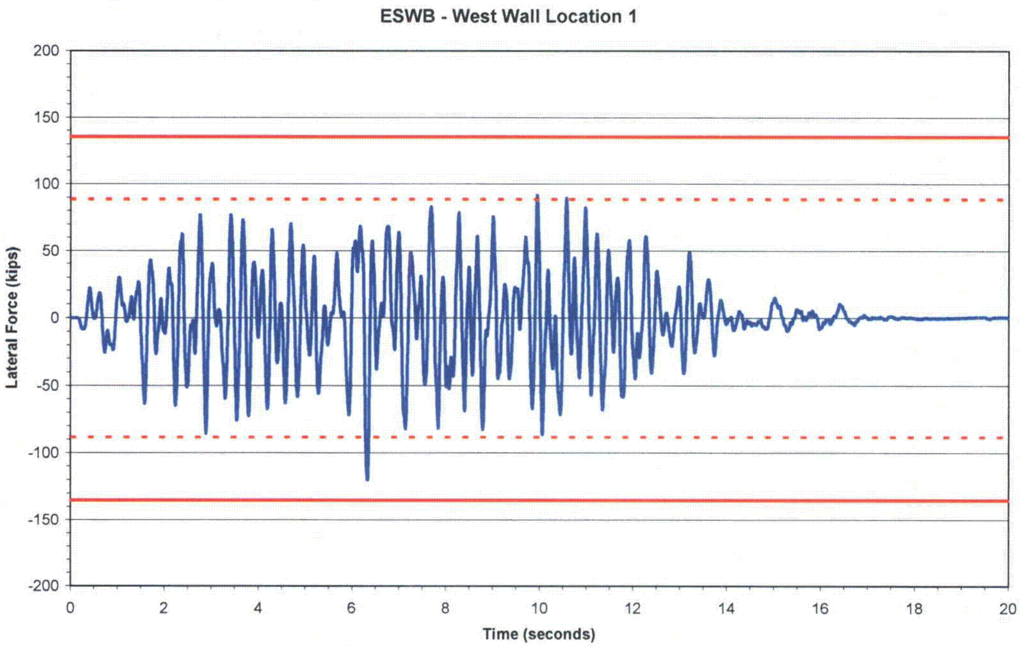
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 26. Lateral Force at South Basement Wall Location 5, ESWB



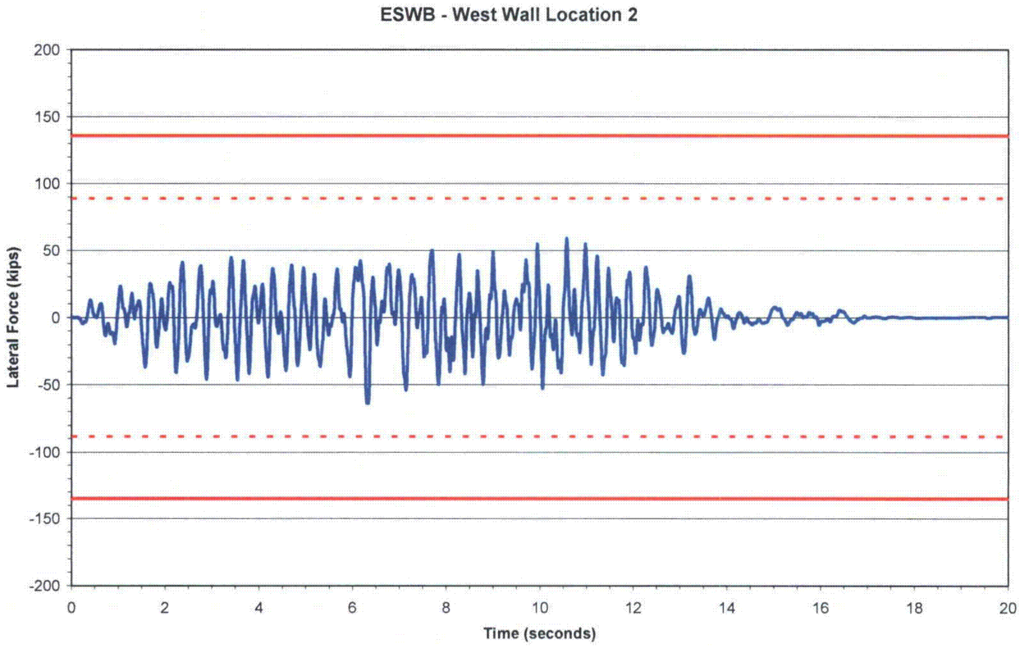
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 27. Lateral Force at West Basement Wall Location 1, ESWB



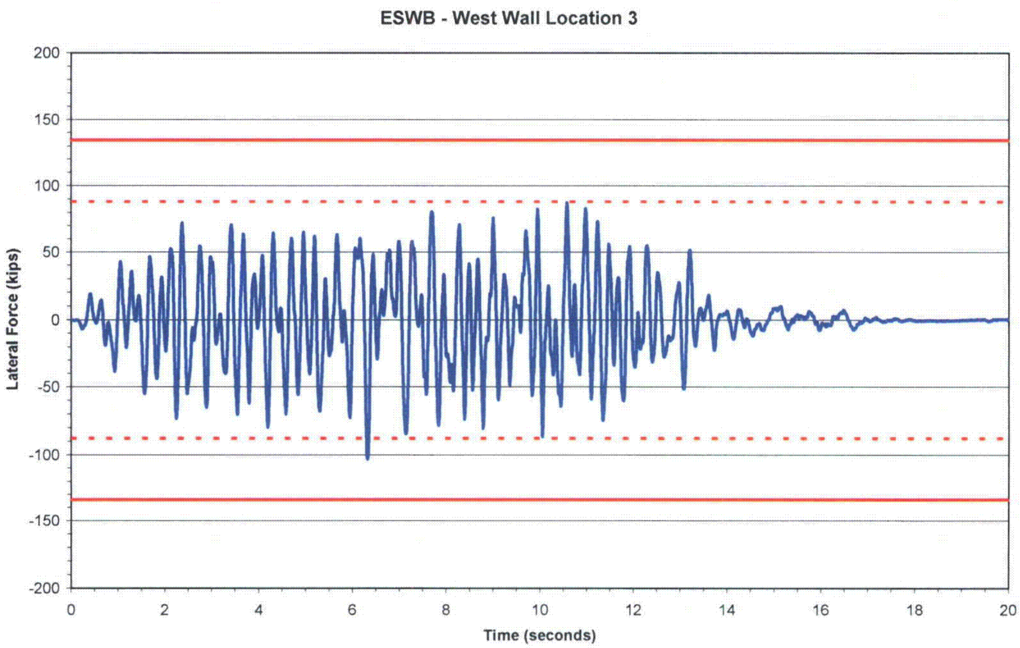
Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 28. Lateral Force at West Basement Wall Location 2, ESWB



Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Figure 29. Lateral Force at West Basement Wall Location 3, ESWB



Solid Red Line= Upper Bound; based on parameters used for the structural design of embedded walls.
Dotted Red Line= Estimated Lower Bound for Reference.

Part 2, Common Basemat Intake Structures (CBIS)

This question was asked with respect to the earlier design and analysis that contained a separate UHS Intake Structure and Electrical Building. These two buildings have since been consolidated into a single structure and re-analyzed. The updated structure and analysis were provided to the NRC in UniStar Letter UN#10-285[2] and incorporated into COLA Revision 7. This response is based on the updated configuration and analysis.

Vertical Tensile Stresses

During seismic response of the structure, no vertical tensile force between the bottom of the basemat and the supporting subgrade is observed. The following presents the details of the evaluation for the tensile stress below the foundation:

- For seismic stability evaluation of the CBIS, 3D finite elements with soil properties were introduced to the SASSI model at the bottom of the MWIS foundation (as shown on Figure 30). The additional 3D soil elements were added at selected regions where sliding and overturning are expected to be critical. The selected regions for the additional 3D finite elements are: the corner of the half model, the region below the UHS MWIS, the region under the center of the Forebay, and the region below the CW building.

- After seismic analysis of the CBIS using the UB, LB and BE soil conditions, and excluding stabilizing static compressive stresses, the maximum tensile stresses due to seismic forces between the foundation of the CBIS and the additional finite soil elements were evaluated. It is important to note that maximum tensile stresses during seismic motions can occur at different times for different regions. However, for conservative evaluation of the stability of the CBIS, the maximum tensile stresses were considered simultaneously, regardless of the time they occurred.

- In addition to the SASSI seismic analysis, the stabilizing compressive static stresses at the foundation of the CBIS were computed from a static PLAXIS 3D analysis. Finally, the seismic tensile stresses obtained from SASSI were superimposed with the stabilizing static compression stresses obtained from PLAXIS 3D in order to obtain the resultant tensile stresses at the base of the CBIS. Tables 1, 2, and 3 show the solid elements considered at the base of the CBIS (shown on Figure 30) together with the maximum tensile stresses from the seismic SASSI analysis and the compression stresses obtained from the PLAXIS 3D analysis. The results show that the stabilizing static compressive stresses are larger than the seismic tensile stresses at all selected regions below the CBIS basemat foundation. This result confirms that no tensile stress is observed between the foundation of the CBIS and the supporting soil.

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[2] G. Gibson (UniStar Nuclear Energy) to Document Control Desk ((NRC), "Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 253, Seismic System Analysis," letter UN#10-285 dated November 16, 2010.

Table 1
Vertical Stresses on Solid Elements under the Basemat of the UHS MWIS Area

Solid Elements	Max. Seismic Tensile stresses from SASSI (ksf)	Static compression stress from PLAXIS 3D (ksf)	Evaluation of Tensile Stress
18	5.41	10.50	No Tension
17	2.06	10.50	No Tension
8	1.62	7.70	No Tension
10	1.32	5.30	No Tension
9	1.62	4.90	No Tension
7	2.12	4.00	No Tension
6	10.30	12.00	No Tension
Average	3.49	7.84	No Tension

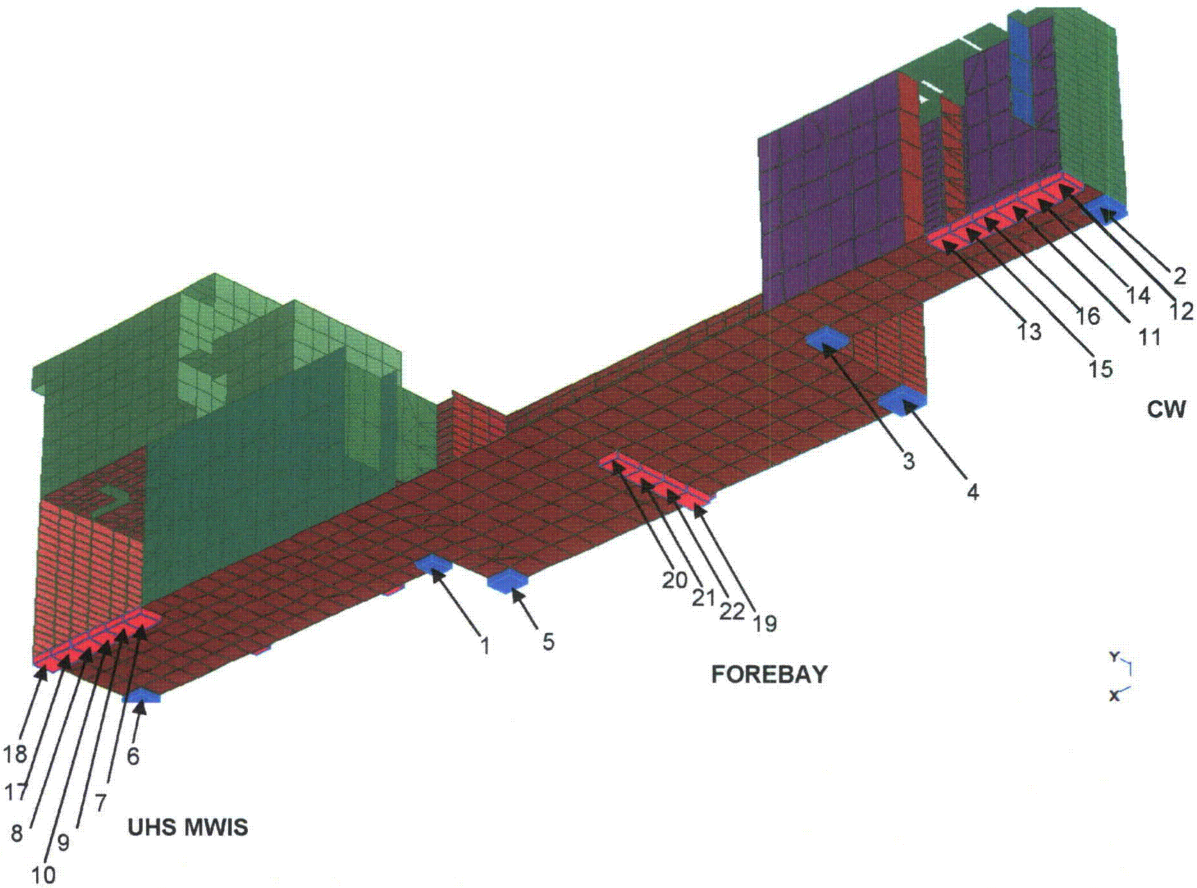
Table 2
Vertical Stresses on Solid Elements under the Basemat of the Forebay Area

Solid Elements	Max. Seismic Tensile stresses from SASSI (ksf)	Static compression stress from PLAXIS 3D (ksf)	Evaluation of Tensile Stress
19	2.14	5.50	No Tension
22	2.01	4.80	No Tension
21	1.33	3.75	No Tension
20	1.31	3.25	No Tension
1	3.64	4.00	No Tension
3	2.38	3.30	No Tension
4	5.28	6.50	No Tension
5	5.76	6.75	No Tension
Average	2.98	4.73	No Tension

Table 3
Vertical Stresses on Solid Elements under the Basemat of the CW Area

Solid Elements	Max. Seismic Tensile stresses from SASSI (ksf)	Static compression stress from PLAXIS 3D (ksf)	Evaluation of Tensile Stress
12	3.22	5.50	No Tension
14	1.78	5.50	No Tension
11	2.28	5.30	No Tension
16	2.06	4.90	No Tension
15	1.09	4.00	No Tension
13	1.06	3.50	No Tension
2	5.05	5.50	No Tension
Average	2.36	4.89	No Tension

Figure 30
Isometric View of the Western Half of the CBIS with Additional 3D Finite Elements for the Seismic Stability Evaluation



Effect of 25 % of the Design Live Load and 75 % of the Design Snow Load

According to NUREG-0800 Section 3.8.5, Acceptance Criteria 3 the load combination for seismic stability evaluation for sliding and overturning is:

D+ F + H + E

However, the seismic stability evaluation performed for the SSI analysis of the CCNPP Unit 3 intake structure considers load combinations used for the SSI analysis which contain 25% of the live load and 75% of the snow load.

D + F+0.25(L) + 0.75(S) + H + E

where D is the Dead Load of the structure and permanent equipment, F is the hydrodynamic pressure, L is the Live Load, S is the Snow load, H is the earth pressure and E is the seismic load.

This effect of the additional 25% of the live load and 75% of the snow load is negligible (less than 1.2%), compared to the total dead load of the structure and permanent equipment as shown in Table 4. Because of the negligible effect of the snow and live loads included in the seismic stability analysis, a separate SSI simulation corresponding to seismic stability load combinations was not performed. Thus, results from the SSI SASSI simulations that contain the 25% of the live load and 75% of the snow load are used for the seismic stability evaluation of the intake structure. However the forces that result from this load combination will be equivalent.

Table 4
Comparison of Loads Considered in the Seismic Stability Analysis of CBIS

Sr. No.	Loads in SSI Analysis	Mass (kips)	Total Mass (kips)	Percentage of live and snow loads
1	Self weight of Intake structure	53,422		
2	Dead load of Plant	1,429		
3	Water in Forebay	13,541		
Total of 1, 2,and 3			68,392	
4	25% of Live load	389		
5	75% of Snow load	426		
Total of 4 and 5			815	
Percentage of live and snow load considered is				1.19 %

Static and Dynamic Lateral Earth Pressures

For the seismic stability evaluation of the CBIS, the static and dynamic earth pressures along the embedment depth were not considered in the sliding and overturning factor of safety computation. The seismic stability evaluation was performed using the dynamic and static stresses only at the interface between the foundation basemat and the soil.

For the seismic stability evaluation, the dynamic stresses at the base of the basemat were obtained from seismic SSI (SASSI) analyses which consider the full embedment of the CBIS. The static stresses at the base of the basemat were obtained from 3D static analysis using PLAXIS 3D, which also includes the full embedment of the CBIS.

COLA Impact

FSAR Subsection 3.7.2.14.2 will be revised as follows:

3.7.2.14.2 EPGB and ESWB

The stability of the EPGB and ESWB for seismic loading is determined using the stability load combinations provided in NUREG-0800 Section 3.8.5, Acceptance Criteria 3 (NRC, 2007a).

For determination of seismic stability, the overturning moments about each of the four edges of the basemat and sliding forces at the bottom of the basemat are computed by using the response time histories of reactions at the basemat nodes. These responses include the effects of seismic forces, ~~static and~~ dynamic lateral earth pressures, and hydrostatic and hydrodynamic forces. The following steps are used to assess the seismic stability:

RAI 253

Question 03.07.02-50

In its response, the applicant states that interior walls carry impulsive pressure on both sides of the wall as shown in Figure 2 included in the response. In evaluating the net pressure distribution on an interior wall, for a given direction of earthquake excitation, the hydrodynamic pressure increases on one side of a wall and decreases on the other side by the impulsive force that is generated within each chamber. Thus there is a net increase on the hydrodynamic load acting on any given wall. The applicant is requested to confirm that this effect is considered in the wall design and include its description in the FSAR. In addition the applicant states on page 3-38 that the entire water mass is lumped at the basemat when determining the seismic response in the vertical direction. The hydrodynamic pressure of the water mass acting on the sides of the wall needs to be increased to reflect its acceleration in the vertical direction. The applicant is requested to revise the FSAR to state that this effect is also considered in the wall design. The information requested will assist the staff's analysis to conclude that the effects of fluid-structure interaction have been properly accounted for in the seismic design of the building.

Response

This question was asked with respect to the earlier design and analysis that contained a separate UHS Intake Structure and Electrical Building. These two buildings have since been consolidated into a single structure and re-analyzed. The updated structure and analysis were provided to the NRC in UniStar Letter UN#10-285[2] and incorporated into COLA Revision 7. This response is based on the updated configuration and analysis.

Net Pressure on an Interior Wall

The hydrodynamic effects of water contained in the CBIS were considered in accordance with ACI 350.3-06. The impulsive and convective water masses due to horizontal earthquake excitation were calculated using the clear dimensions between the walls perpendicular to the direction of motion and the height of water in the Forebay and in the UHS Makeup Water Intake Structure. The convective water masses were attached to the walls using appropriate spring stiffness calculated in accordance with ACI 350.3-06. The impulsive water masses were attached to the walls in the Forebay and basement walls of the UHS Makeup Water Intake Structure using rigid springs and are included in the SSI model of the CBIS.

From the hydrodynamic loads model, for a given direction of earthquake excitation, the hydrodynamic pressure increases on one side of a wall and decreases on the other side resulting in a net increase on the hydrodynamic load acting on any given wall. This effect was modeled in the SSI model of the CBIS. The design of the CBIS walls includes this effect.

Effect of Vertical Seismic Vibration on Horizontal Hydrodynamic loads

The SSI analysis performed for the CCNPP Unit 3 CBIS considered the effect of the horizontal seismic vibration on the horizontal hydrodynamic forces (convective and impulsive hydrodynamic forces) using ACI 350.3-06 procedures. For vertical vibration, the entire water mass is lumped at the basemat nodes for earthquake ground motion in the vertical direction.

However, the effect of the vertical vibration on the horizontal hydrodynamic forces was not considered.

The effect of the vertical vibration on the horizontal hydrodynamic forces was assessed in a separate calculation. In order to understand the effect of the vertical vibration on the horizontal hydrodynamic loads, the horizontal hydrodynamic forces and moments due to vertical vibration were computed on an internal wall in the UHS MWIS and compared with the horizontal hydrodynamic forces due to horizontal vibration of the CBIS (impulsive and convective forces).

The horizontal hydrodynamic pressure distribution on a vertical wall due to vertical vibration acceleration a_v is:

$$P = \rho a_v h$$

Where P is the horizontal pressure on the wall, ρ is the density of water and h is the depth of water below the water surface. This expression is similar to the static water pressure distribution, except that the gravitational acceleration (9.81 m/s) is replaced by the seismic vertical acceleration a_v. Hence, the water pressure has a linear distribution with the minimum and maximum values at the top and bottom of the tank respectively.

To compare the magnitude of the horizontal hydrodynamic load due to vertical and horizontal seismic vibration, the corresponding forces and bending moments are computed on an internal wall A-B inside the UHS MWIS room below grade.

Figure 31 shows a schematic view of the western half of the CBIS basemat floor, which was considered in the SSI analysis of the CBIS. Figure 32 shows an isometric view of the detail of the UHS MWIS walls subjected to hydrodynamic loads. Figures 33 and 34 show schematic views of the water storage area inside the UHS MWIS, together with the convective and impulsive hydrodynamic nodes and springs in the East-West direction. The SASSI simulation results, which correspond to the upper bound (UB) soil with 5% critical damping ratio, were used to get the maximum accelerations at the location of the hydrodynamic mass nodes and at the basemat floor.

Table 5 summarizes the bending moment acting at the base of the internal UHS MWIS wall A-B from horizontal hydrodynamic forces due to horizontal and vertical seismic vibrations. The results in Table 5 show that the horizontal hydrodynamic forces from horizontal seismic vibration (convective and impulsive loads) are much larger than the horizontal hydrodynamic loads from vertical seismic vibration.

Hence, the contribution of vertical seismic excitation on horizontal hydrodynamic loads acting on walls is extremely small (less than 2%) and does not have any significance on the design. This leads to the conclusion that the effect of the vertical vibration on the horizontal hydrodynamic loads is negligible and does not impact the design of the CBIS walls.

COLA Impact

There are no changes to the CCNPP Unit 3 COLA as a result of this response.

Table 5
Comparison of Horizontal Hydrodynamic Forces Acting on UHS Internal Wall A-B from Horizontal Vibration and the Vertical Vibration for the Upper Bound Soil Condition

UHS ROOM	Moment from Convective and Impulsive Hydro. Dyn. Forces (Kips-ft)	Moment from Vertical Excitation (kips-ft)	Percentage of Vertical Excitation Effect %
ROOM 1	655.03	6.99	1.07
ROOM 2	420.44	3.18	0.76
ROOM 3	117.12	2.26	1.93

Figure 31
Schematic View of Western Half of CBIS Foundation Floor

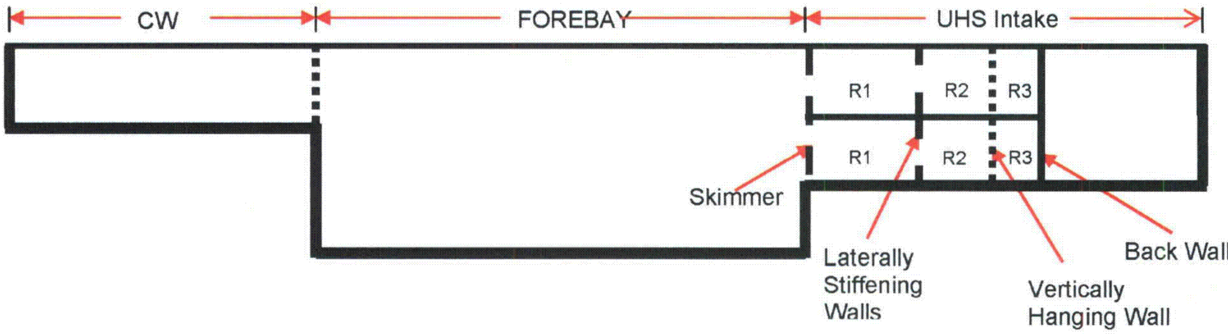


Figure 32
Isometric View of the Quarter Model of the CBIS Showing Detail of the Water Storage Room and Walls Inside the UHS MWIS

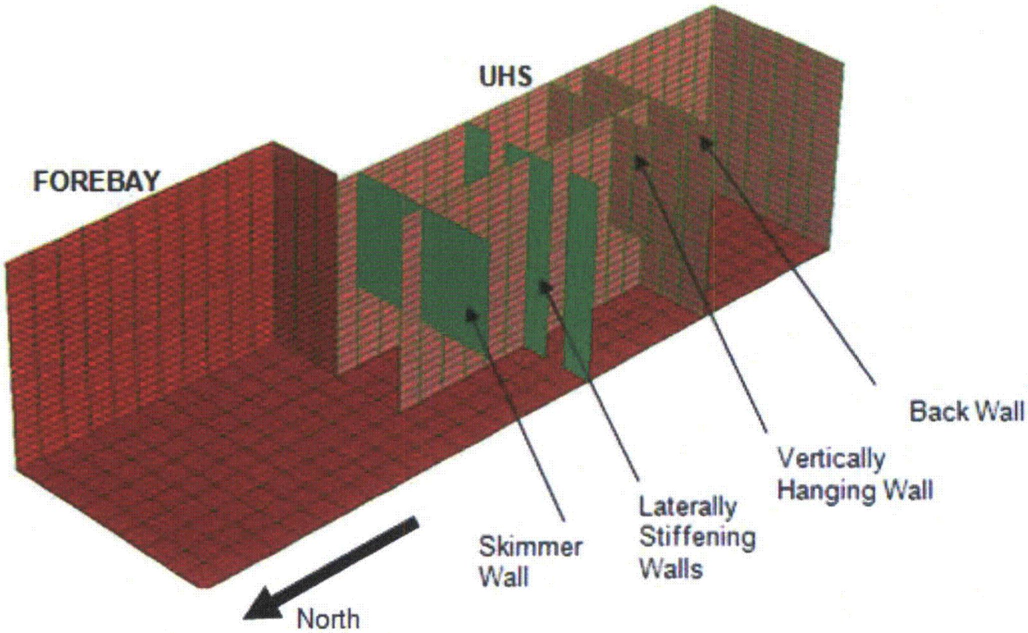


Figure 33
Schematic View of East-West Direction Convective Hydrodynamic Masses and Springs Inside UHS MWIS First Floor Rooms
(To Compute Forces on Internal Wall A-B)

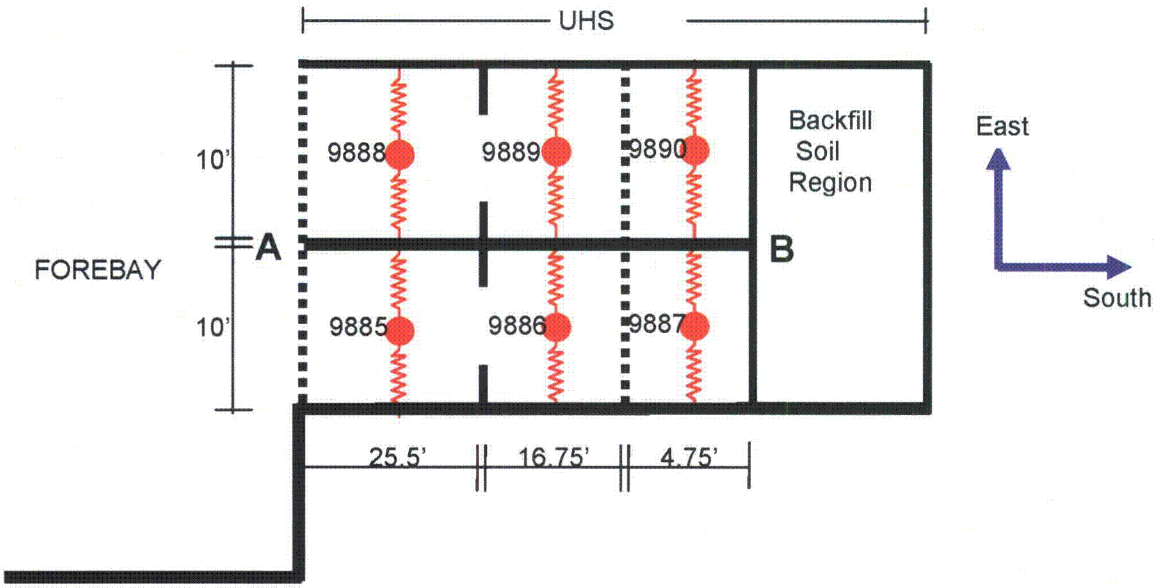
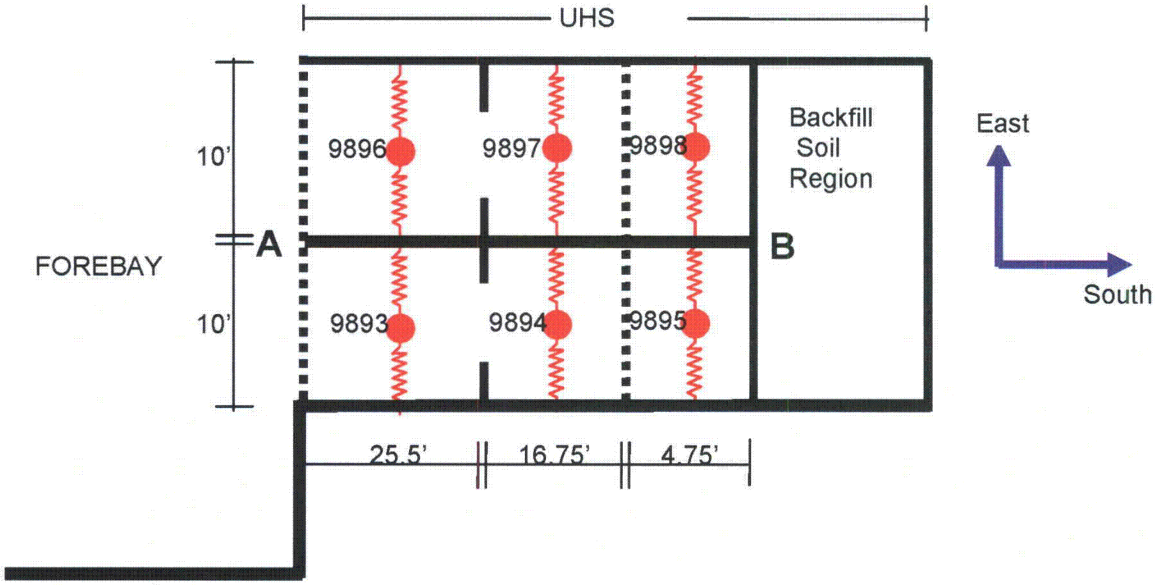


Figure 34
Schematic View of East-West Direction Impulsive Hydrodynamic Masses and Springs Inside UHS MWIS First Floor Rooms
(To Compute Forces on Internal Wall A-B)



RAI 253

Question 03.07.02-51

Section 3.7.2.3.2 (page 3-38) indicates that the thick shell element formulation (SHL17) is used in the SASSI SSI calculations for the Common Basemat Intake Structure to account for transverse shear deformation effects as well as in-plane and out-of-plane effects. Given the thickness of the walls and slabs of the structure relative to the spans, it appears that the use of a thick shell element is appropriate. However, the applicant's response indicates that the GTSTRUDL static model made use of a thin shell element (SBHQ6), which does not consider transverse shear effects. The applicant is requested to provide an evaluation of the differences in calculated forces and moments which occur between using the SBHQ6 element vs. using an element that includes shear deformation effects. The use of the SBHQ6 element may produce non-conservative results when determining the distribution of forces and moments within the structure with the result that the structure may be under-designed for Safe Shutdown Earthquake loads.

Response

This question was asked with respect to the earlier design and analysis that contained a separate UHS Intake Structure and Electrical Building. These two buildings have since been consolidated into a single structure and re-analyzed. The updated structure and analysis were provided to the NRC in UniStar Letter UN#10-285[(2)] and incorporated into COLA Revision 7. This response is based on the updated configuration and analysis.

New calculations were performed to consider the new location of the Electrical Building on top of the Ultimate Heat Sink (UHS). Different software was used for these new analyses compared to the previous effort. STAAD Pro Version 8i was used for static structural analysis and RIZZO SASSI version 1.3a was used for seismic soil-structure interaction analyses. The plate element in STAAD Pro Version 8i includes the effect of the shear deformation in the stiffness formulation; therefore, it can be applied to thin and thick slabs and walls. On the other hand, the plate element of RIZZO SASSI v1.3a considers thin plate element formulation and does not include shear deformation. However, the thin plate element formulation in RIZZO SASSI v1.3a is considered appropriate to model the CCNPP3 CBIS based on the recommendations of ASCE 4-98. According to ASCE 4-98, the effect of shear flexibility should be included when the ratio thickness/distance between inflexion point is larger than 0.5. Figures 35 through 39 show the thickness/distance between inflection points ratio calculated for the walls and slabs of the CW, UHS, EB and Forebay. These values are smaller than 0.5, which support the use of the thin plate element of RIZZO-SASSI for the CBIS structure according to ASCE 4-98.

Note that the spans of the slabs are considered as the distance between inflection points as shown in Figures 35 through 37 for the UHS MWIS and CW. The reason for this selection is that out-plane bending in the slabs is mobilized when the earthquake acts in the vertical direction, whereas the acceleration increases around the center of the slab with inflection points located close to the support provided by the walls.

Bending of the walls about the vertical and horizontal axes was also revised. In the case of bending about the vertical axis, the deformation in the walls occurs along the horizontal direction, so the span of the wall is equal to the distance between the two perpendicular walls resisting the bending, which is also the span of the slab supported on the wall. Hence, the spans of the walls bending about the vertical axis are equal to the values considered for the slabs shown in Figures 35 through 37. On the other hand, the thickness of any wall in this model is always smaller than the thickness of the slab located at the bottom of the wall. Therefore, the thickness/span ratios shown in Figures 35 through 37 for the slabs bound the values for walls subjected to bending about the vertical axis. The values of thickness/span ratio for the Forebay wall bending about the vertical axis are shown in Figure 38.

For walls bending about the horizontal axis, a conservative approach was followed where the distance between inflection points is considered as half of the story height. These thickness/span ratios are illustrated in Figures 38 and 39 for the Forebay and UHS MWIS, respectively.

COLA Impact

There are no changes to the CCNPP Unit 3 COLA as a result of this response.

Figure 35
Floor thickness/Span length ratio for basemat at Elevation -22'-6"

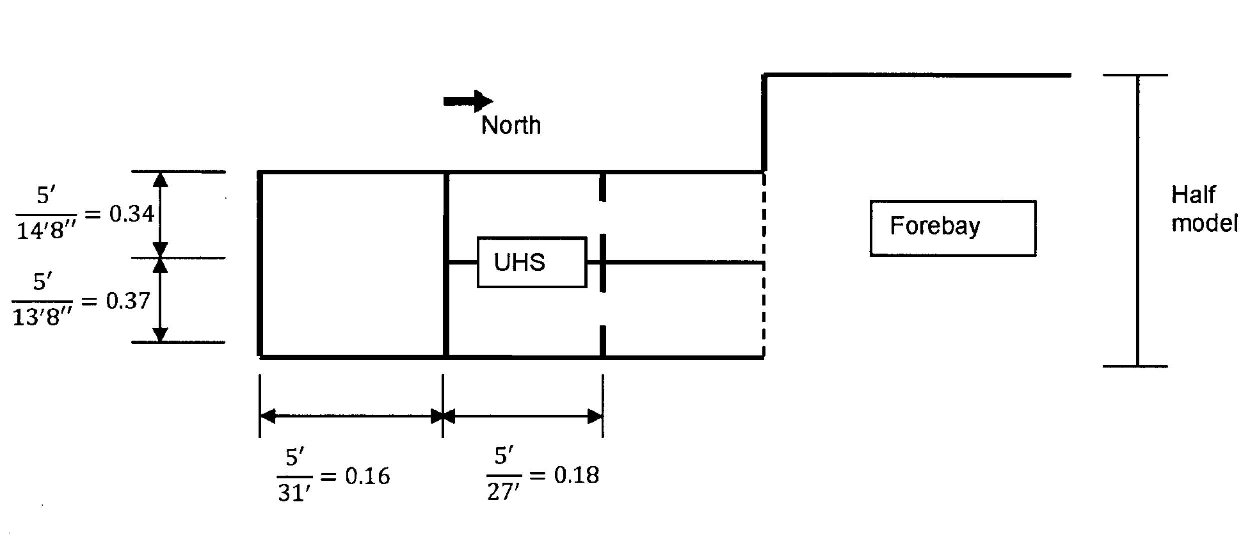
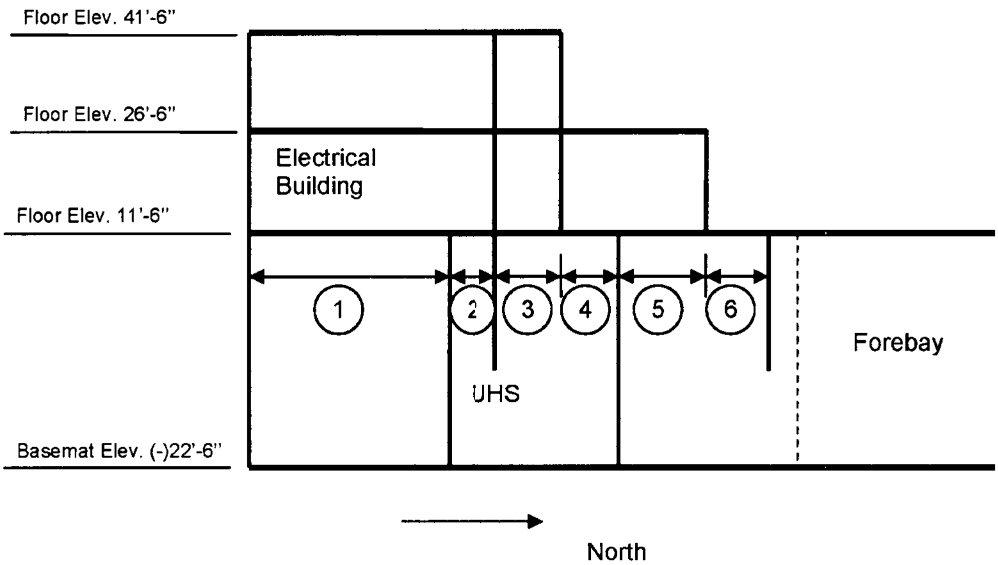


Figure 36
Floor Thickness/Span Length Ratio for Slabs In the UHS MWIS at Elevation 11'-6"



1 $\frac{4'}{31'} = 0.13$

2 $\frac{3'}{8'} = 0.38$

3 $\frac{3'}{10'} = 0.3$

4 $\frac{3'}{10'} = 0.30$

5 $\frac{3'}{13'6''} = 0.22$

6 $\frac{3'}{10'} = 0.3$

Figure 37
Floor Thickness/Span Length Ratio for Slabs Located in the CW Building

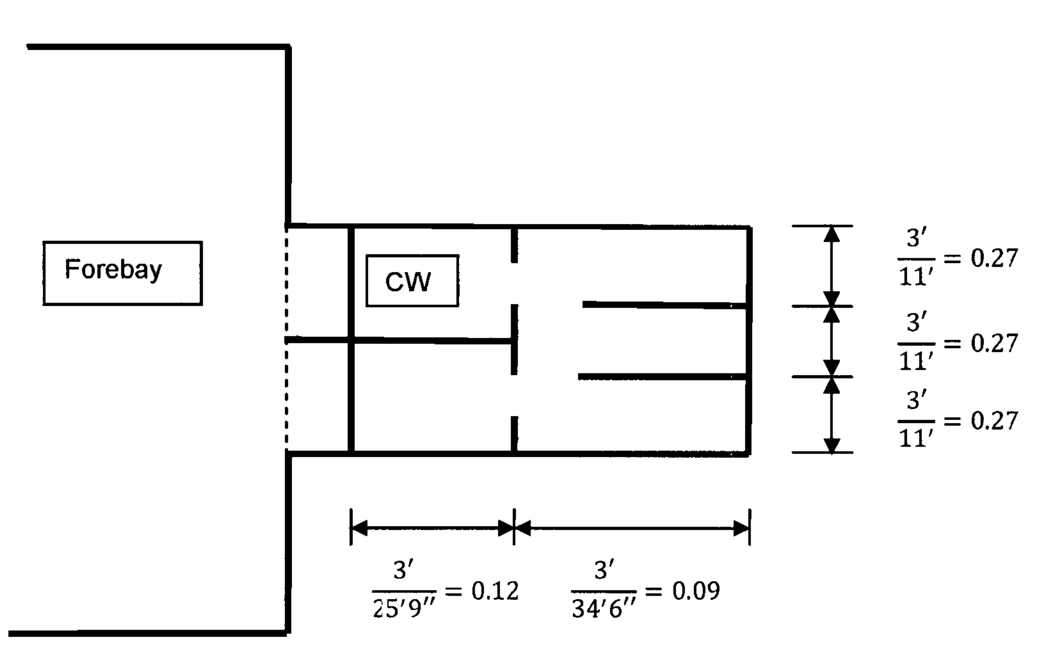


Figure 38
Thickness/ Distance Between Inflexion Points Ratio for the Forebay Walls Bending About Vertical and Horizontal Axis

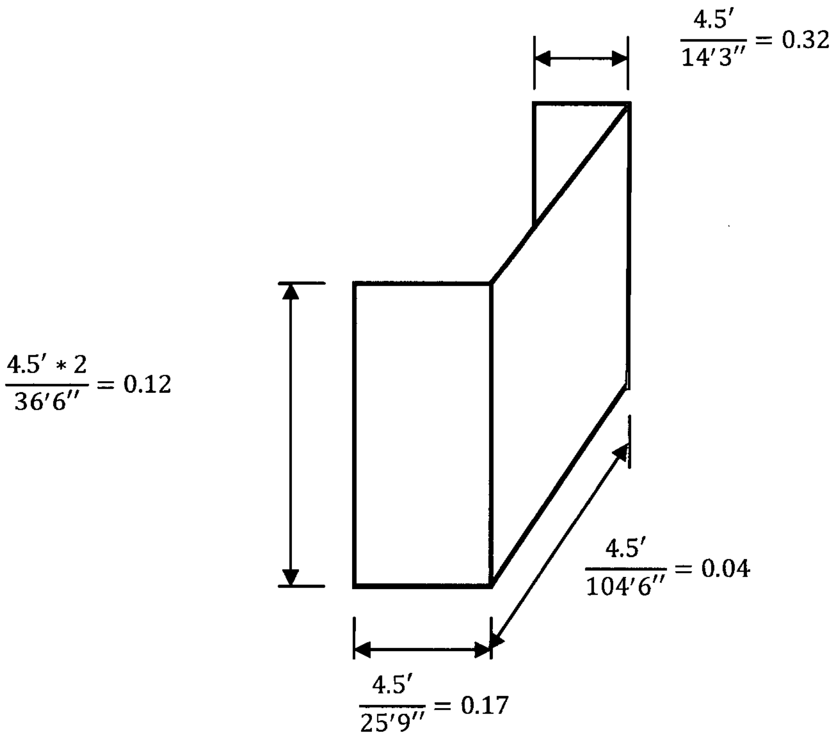
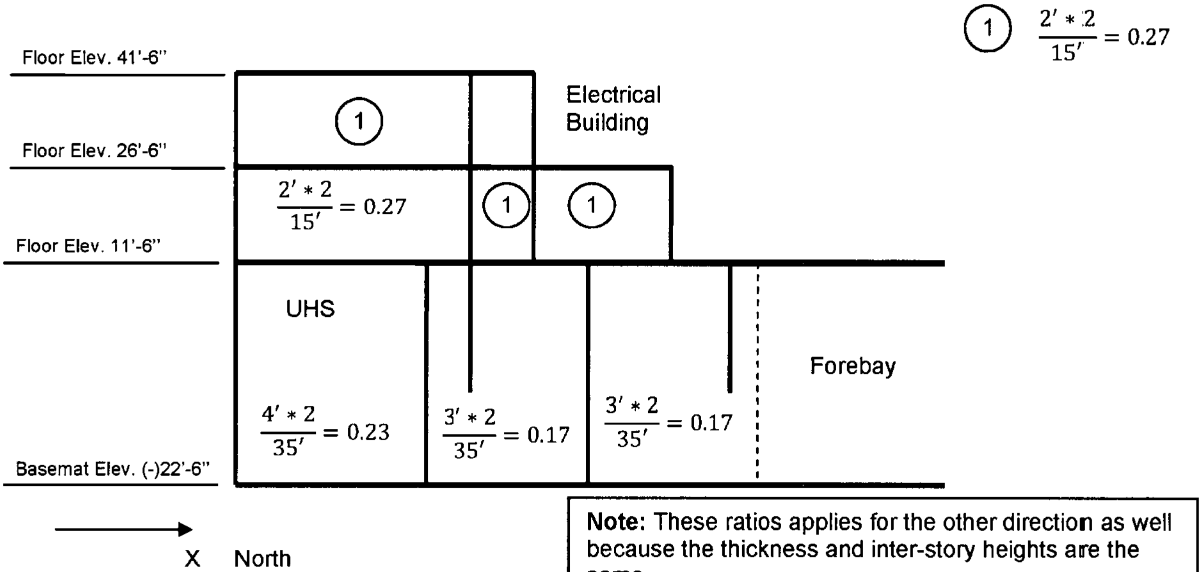


Figure 39
Thickness/Distance Between Inflexion Points Ratio for the Walls in the UHS MWIS Area



Note: These ratios applies for the other direction as well because the thickness and inter-story heights are the same