Nuclear Operating Company

South Texas Project Electric Generating Station P.O. Box 289 Wadsworth, Texas 77483

September 15, 2010
U7-C-STP-NRC-100208

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
One White Flint North
11555 Rockville Pike
Rockville, MD 20852-2738

South Texas Project
Units 3 and 4
Docket Nos. 52-012 and 52-013
Response to Request for Additional Information

Attached are the responses and revised responses to NRC staff questions included in Request for Additional Information (RAI) letter numbers 349, 350, and 358 related to Combined License Application (COLA) Part 2, Tier 2, Sections 3.4, 3.7 and 3.8. The attachments address the new or revised responses to the RAI questions listed below:

03.04.02-1	03.08.01-9
03.04.02-6	03.08.01-10
03.04.02-10	03.08.04-18
03.04.02-11	03.08.04-22
03.07.01-25	03.08.04-28
03.07.01-26	03.08.04-29
03.07.01-27	03.08.04-31
03.07.01-28	03.08.04-32
03.07.02-23	03.08.04-33
03.07.02-24	03.08.05-4
03.08.01-4	03.08.05-5
03.08.01-7	

There are no commitments in this response.

Where there are COLA markups, they will be made at the first routine COLA update following NRC acceptance of the RAI response.

If you have any questions regarding these responses, please contact Scott Head at (361) 972-7136, or Bill Mookhoek at (361) 972-7274.

D091
NRO

STI 32747004

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

Executed on 9/15/10

Scott Head
Manager, Regulatory Affairs
South Texas Project Units 3 & 4

jep

Attachments:
1. 03.04.02-1, Rev 1
2. 03.04.02-6, Rev 1
3. 03.04.02-10
4. 03.04.02-11
5. 03.07.01-25
6. 03.07.01-26
7. 03.07.01-27
8. 03.07.01-28
9. 03.07.02-23
10. 03.07.02-24
11. 03.08.01-4, Rev 1
12. 03.08.01-7, Rev 1
13. 03.08.01-9
14. 03.08.01-10
15. 03.08.04-18, Rev 1, Supp 1
16. 03.08.04-22, Rev 1
17. 03.08.04-28
18. 03.08.04-29
19. 03.08.04-31
20. 03.08.04-32
21. 03.08.04-33
22. 03.08.05-4
23. 03.08.05-5

cc: w/o attachment except*
(paper copy)

Director, Office of New Reactors
U. S. Nuclear Regulatory Commission
One White Flint North
11555 Rockville Pike
Rockville, MD 20852-2738

Regional Administrator, Region IV
U. S. Nuclear Regulatory Commission
611 Ryan Plaza Drive, Suite 400
Arlington, Texas 76011-8064

Kathy C. Perkins, RN, MBA
Assistant Commissioner
Division for Regulatory Services
Texas Department of State Health Services
P. O. Box 149347
Austin, Texas 78714-9347

Alice Hamilton Rogers, P.E.
Inspection Unit Manager
Texas Department of State Health Services
P. O. Box 149347
Austin, Texas 78714-9347

*Steven P. Frantz, Esquire
A. H. Gutterman, Esquire
Morgan, Lewis & Bockius LLP
1111 Pennsylvania Ave. NW
Washington D.C. 20004

*Tom Tai
Two White Flint North
11545 Rockville Pike
Rockville, MD 20852

(electronic copy)

*George F. Wunder
*Tom Tai
*Tekia Govan
Loren R. Plisco
U. S. Nuclear Regulatory Commission

Steve Winn
Joseph Kiwak
Eli Smith
Nuclear Innovation North America

Peter G. Nemeth
Crain, Caton & James, P.C.

Richard Peña
Kevin Pollo
L. D. Blaylock
CPS Energy

RAI 03.04.02-1, Revision 1

QUESTION:

The STP applicant incorporated ABWR DCD, Section 3.4.2, Revision 4, by reference with departures including STP DEP T1 5.0-1. The departure introduces a new set of site-specific loads including hydrodynamic loads not accounted for within the certified scope of ABWR DCD. Discuss the site specific flood (maximum flood level is 1478.3 cm above MSL) design issues including how the lateral hydrodynamic pressure on the structures due to the design flood water level, as well as ground and soil pressures, are calculated. Also, to the extent IBC 2006, which references ASCE 7-05, is adopted at STP Units 3 and 4, justify its application for the flood design of STP SSCs.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-090161, dated October 7, 2009. This response is being revised as a result of the changes identified in the response to RAI 03.04.02-11 which is being submitted concurrently with this response. This revised response completely supersedes the original response. The revised portions of the response are marked with revision bars.

As provided in COLA Part 2, Tier 2, Section 3H.2.4.2.3, the design basis flood level was revised to 182.9 cm (6 ft) above grade. The nominal plant grade is at elevation 34 ft.

The following is based on the Main Cooling Reservoir (MCR) embankment breach analysis results provided in COLA Part 2, Tier 2, Section 2.4 and the response to RAI 03.04.02-11:

- Maximum calculated water level near the safety-related structures is at elevation 38.8 ft. Design flood level is conservatively established at elevation 40 ft.
- Maximum water velocity is 4.72 ft/sec.
- Maximum hydrodynamic drag force due to flood water flow is 44 pounds per square foot of the projected submerged area. This hydrodynamic load is in accordance with Section 5.4.3 of ASCE 7-05 using a conservative drag coefficient of 2.0.
- Hydrodynamic forces due to wind generated wave forces are as shown in Figure 3.4-1 which is provided in the response to RAI 03.04.02-11.
- Debris loading consists of impact due to a 500 lbs floating debris traveling at 4.72 ft/sec.

This revised design basis flood level will impact the following:

- Design of exterior walls of the Reactor Building (RB) and Control Building (CB), both above and below grade
- Flotation safety factor of the RB and CB
- Flood protection of the RB and CB against external flooding
- The hydrostatic head for design of seals at seismic gaps and penetrations

- Design of non-safety-related SSCs to withstand the design basis flood in order not to impair safety functions of the adjacent safety-related SSCs

The impact on the design of exterior walls of the RB and CB and the impact on flotation safety factor of the RB and CB are provided in the responses to RAI 03.08.01-4 Revision 1 and RAI 03.08.01-7 Revision 1, being submitted concurrently with his response.

The impact on flood protection of the RB and CB against external flooding is provided in the response to RAI 03.08.01-9 and RAI 03.04.02-6 Revision 1, being submitted concurrently with this response.

The impact on the design of seals at seismic gaps and penetrations is provided in the response to RAI 03.04.02-5.

The impact on the design of non-safety-related SSCs to withstand the design basis flood in order not to impair the safety functions of the adjacent safety-related SSCs is provided in the response to RAI 03.04.02-10, being submitted concurrently with this response.

Flood protection, design, and stability safety factors of the site-specific safety-related SSCs are based on the revised design basis flood level.

No additional COLA change is required for this response.

RAI 03.04.02-6, Revision 1

QUESTION:

In its evaluation of **RAI 03.04.02-2 (ID 3322 Question 13162)**, the staff accepts in general the applicant's physical description of watertight door locations and the proposed measures and procedures to accomplish water tightness of any below DBFL openings and penetrations of seismic category I, in-and out-of-scope SSC, as reflected in the proposed revision to COLA FSAR. The staff considers that since watertight doors are seismic category I SSC, each exterior door under DBFL located in any category I structure should be given a unique component ID, a set of specific design parameters, other conditions (e.g., controls measures) and be keyed into the corresponding plans to show each door's location. Such information should be reflected in the ITAAC tables conveying the design requirements, the proposed inspections, tests, analyses and the acceptance criteria including the need for as-built reconciliation which is required for category I SSC. All certified and plant-specific category I SSC should be considered, including the underground diesel tanks and vaults if applicable. Compliance with RG1.102 Flood Protection for Nuclear Power Plants should also be indicated for the underground diesel tank access openings if applicable. The staff needs this information to be able to conclude that the seismic category I doors are designed and installed to withstand the design basis flood during an accident.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100154, dated June 29, 2010. The original response is completely superseded by this revised response. The revisions are indicated by revision bars in the margin.

Each of the exterior watertight doors used for protection against a Design Basis Flood (DBF) will be given a unique component ID. The specific design parameters and other conditions will be contained in the purchase specification for the doors, and are included in the COLA markups included with this response. The design commitments, as-built reconciliation requirements, required inspections, tests, analyses and acceptance criteria for penetrations in exterior walls below design basis flood level are included in ITAAC Tables 2.15.10 and 2.15.12. ITAAC Table 2.15.10 also applies to the watertight doors in the Diesel Generator Fuel Oil Storage Vaults. The ITAACs for both the Reactor Building (Table 2.15.10) and the Control Building (Table 2.15.12) state that "Penetrations in the external walls below flood level are provided with flood protection features." The ITAACs for both buildings state that they are protected from external flooding events and require a Flood Analysis Report that includes the results of inspections of the as-built flood protection features.

This RAI response will impact previously submitted responses to the following RAIs and COLA Sections:

- COLA Part 2, Tier 2 Sections 3.4.3.1 and 3.4.3.3
- COLA Part 2, Tier 2 Section 2.4S.10
- RAI 03.04.02-2
- RAI 03.08.01-3
- RAI 03.08.01-6
- RAI 14.03.02-9
- RAI 19-30

The markup to the COLA Sections is presented in Enclosure 1 and the revised RAI responses are being submitted concurrently with this response. The COLA markups include the description of loads, load combinations, and acceptance criteria for the watertight doors. Please note that Section 3H.6.7, which is referenced in the revision to Section 3.4.3.3, was submitted with response to RAI 03.07.01-19 Revision 2, as submitted in STPNOC letter U7-C-STP-NRC-100129, dated June 7, 2010.

Enclosure 2 presents the COLA markup for COLA Part 2, Tier 2, Section 2.4S.10, as requested by NRC during the site audit on August 31 – September 1, 2010.

RAI 03.04.02-6, Revision 1
Enclosure 1
Revisions to COLA Part 2, Tier 2, Sections 3.4.3.1 and 3.4.3.3

3.4.3.1 Flood Elevation

The following paragraphs will be added at the end of this section

Watertight doors or barriers are provided on the Reactor Building and Control Building to protect the buildings from the external design basis flood. These watertight doors or barriers are considered Seismic Category I components. In order to ensure that the watertight doors and barriers can withstand the ABWR Standard Plant loading requirements, the watertight doors and barriers of the Reactor Building and Control Building will be designed for the more severe of the standard plant and site-specific loading. Watertight doors shall be designed to meet the Incorporated Barrier requirements of Regulatory of Guide 1.102 *Flood Protection for Nuclear Power Plants.*

The watertight doors or barriers for the Reactor Building consist of the six exterior doors and the exterior Large Equipment Access indicated in Tier 1 Figures 2.15.10h and 2.15.10j. The watertight doors for the Control Building consist of the access doors between the Control Building and the Service Building shown in Tier 1 Figures 2.15.12d, e, and f, the exterior equipment access door shown in Tier 1 Figure 2.15.12g, and an access door between the Control Building and the Service Building shown in Tier 1 Figure 2.15.12g. Each door will be given a unique component ID in the construction drawings.

The locations for watertight doors in the Reactor Building and Control Building include:

Exterior Watertight Door or Barriers

Structure	Door or Barrier Description	Elevation
Reactor Building	Clean Access Area Corridor Entrance	B1F (4800 mm)
	Diesel Generator A Access	1F (12300 mm)
	Diesel Generator B Access	1F (12300 mm)
	Diesel Generator C Access	1F (12300 mm)
	East Equipment Hatch Access	1F (12300 mm)
	West Equipment Hatch Access	1F (12300 mm)
	Large Equipment Access	1F (12300 mm)
Control Building	HX Area Access at Service Building	B3F (-2150 mm)
	Electrical Area Access at Service Building	B2F (3500 mm)
	Control Building Access at Service Building	B1F (7900 mm)
	Entrance to Reactor Building Controlled Access	1F (12300 mm)
	Equipment Access	1F (12300 mm)

The watertight doors are seated such that the force of the water helps maintain the watertight seal. The watertight doors are designed to be leak tight. Watertight doors will be individually engineered assemblies designed by the supplier to satisfy the design basis performance requirements for external flooding. Watertight doors will allow only slight seepage during an external flooding event in accordance with criteria for Type 2 closures in U. S. Army Corps of Engineers (COE) EP 1165-2-314, "Flood-Proofing Regulations". This criterion will be met under hydrostatic loading of 12 inches of water above the design basis flood elevation per Table 3.4-1, plus drag effects, as required. Water retaining capability of the doors shall be demonstrated by qualification tests for the water head levels. These tests will be completed prior to shipment of the doors. For this purpose a test fixture may be used, with gasket material and cross section, its retainers, and the anvil configuration being identical to that of the full size doors. The test fixture shall have the necessary valving, pressure gages, flow meters, and instruments for measuring gasket compression. To validate that the door satisfies a Type 2 closure per (COE) EP 1165-2-314, the leakage shall not exceed 0.10 gallon/hour/linear foot of gasket when subjected to 125% of the specified head pressure. The hydrostatic head shall be raised at a rate not more than 1 ft/min. If leaks occur during the rising of the hydrostatic head and the leakage rate begins to diminish as the hydrostatic head increases, the assembly shall be tested at the hydrostatic head where the more substantial leakage was observed.

The seals between the Reactor Building and the Control Building below the design basis flood level shall be made using a polyurethane foam impregnated with a waterproof sealing compound. The seals shall be tested to be watertight when subjected to the maximum anticipated hydrostatic head at movements of +/-25% of the designed gap size to demonstrate that the material is capable of being watertight after the effects of long-term settlement or tilt, as well as during normal operating vibratory loads, such as SRV actuation. Although this will provide margin to accommodate differential displacements from the majority of the movements from short duration extreme environmental loading such as SSE, the seals need not be designed to be watertight during the maximum differential displacements from these extreme environmental loadings.

The seals used to protect the safety-related buildings against external water entry are classified as seismic category I with respect to their ability to remain in-place to stop significant water leakage into the safety-related buildings during and after a seismic event. An in-service inspection program will ensure that the seals do not significantly degrade.

The watertight doors or barriers that are utilized for protection against external flooding are normally closed and are used for egress, as required.

The watertight doors, frames, and all components are designed to the requirements of AISC N690 and SRP Section 3.8.4. The structural steel used for the watertight doors conforms to either ASTM A36, ASTM A992 or ASTM A500 Grade B. The faceplate conforms to ASTM A36 or ASTM A606, type 4 and the rubber gasket conforms to ASTM D1056 Type 2 Class D. Fabrication of the doors shall meet the requirements of AISC N690. The welding shall meet the requirements of nondestructive testing, personnel qualifications and acceptance criteria contained in AWS D1.1.

The watertight doors shall be designed for the following loads and load combinations:

$S = D + W + P_o$

$1.6S = D + E' + P_o$

$1.6S = D + W_t + P_o$

$1.6S = D + FL + P_o$

Where:

S = Normal allowable stresses as defined in AISC N690

D = Dead Loads.
E' = Loads generated by SSE, per Sections 3H.1 and 3H.2.
P_o = Loads due to normal operating differential pressure.
FL = Design basis extreme flood loads, including the hydrostatic load due to flood elevation at 40 ft MSL, the associated drag effects of 44 psf, hydrodynamic load due to wind-generated wave action per Figure 3.4-1, and impact due to floating debris per Section 3.4.2.
W = Normal Wind Loads, per Sections 3H.1 and 3H.2.

W_t = Tornado Loads, per Sections 3H.1 and 3H.2, including wind velocity pressure W_w, differential pressure W_p, and tornado-generated missiles (if not protected) W_m

The value used for W_t shall be computed to satisfy the following possible combinations:

$W_t = W_w$

$W_t = W_p$

$W_t = W_m$

$W_t = W_w + W_m$

$W_t = W_w + 0.5\,W_p$

$W_t = W_w + 0.5\,W_p + W_m$

3.4.3.3 Flood Protection Requirements for Other Structures

The following paragraphs will be added at the end of this section

Watertight doors or barriers are provided on the site-specific Diesel Generator Fuel Oil Storage Vaults to protect the vaults from the external design basis flood. These watertight doors or barriers are considered site-specific Seismic Category I components. Each door will be given a unique component ID in the construction drawings.

The locations of watertight doors for the Diesel Generator Fuel Oil Storage Vaults include:

Exterior Watertight Door or Barrier

Structure	Door Description
Diesel Generator Fuel Oil Storage Vaults	Access to Vault A
	Access to Vault B
	Access to Vault C

The design requirements for Diesel Generator Fuel Oil Storage Vault watertight doors are similar to the requirements described in Section 3.4.3.1, except that only the site-specific loads are considered, as described in Section 3H.6.4.

RAI 03.04.02-6, Revision 1
Enclosure 2
Revisions to COLA Part 2, Tier 2, Section 2.4S.10

The following paragraph will be added to COLA Part 2, Tier 2, Section 2.4S.10:

Safety-related facilities are designed to withstand the combination of flooding conditions and wave-run up, including both static and dynamic flooding forces, associated with the flooding events discussed in Subsection 2.4S. Protection of safety-related structures and components is discussed in Subsection 3.4.

An MCR embankment breach could result in significant erosion of earth material in the area of the breach. If this were to occur in the STP 3 & 4 power block area, the foundations for the safety-related facilities are deep enough to withstand the erosive forces of the MCR embankment breach and would not be affected. The bottom of the safety-related facility foundation elevations range from approximately elevation -50.25 ft MSL for the Reactor Building to approximately elevation 4 ft MSL for the UHS basin. Static and dynamic flood forces for Seismic Category I structures are discussed in Subsection 3.4.

The design requirements for flood are discussed in Section 3.4.2. The watertight doors protect the Seismic Category I structures against the site-specific flooding. The doors are designed as Seismic Category I structures. The details of the design requirements for watertight doors are included in Subsection 3.4.3.1 and 3.4.3.3. Flood protection for the penetrations and accessways is described in Table 3.4-1.

RAI 03.04.02-10

QUESTION:

In its evaluation of Open Item 03.04.02-8, the staff noted that the applicant provided only a partial response to the questions regarding the design of SSC with interaction potential subject to flood and other severe environmental loading. The staff agrees with the following aspects of the applicant's response:

(a) Hydrostatic and hydrodynamic design flood forces would be provided as answer to RAI 03.04.02-9;

(b) Concrete structures would be designed according to ACI 349-97, Section 9.2.1, which provides load combinations including extreme environmental loads such as extreme floods, by substituting Wt (tornado loads) with Fa (flood loads) in load combination number 5; and

(c) For non-Seismic Category I structures with potential for interaction, evidence of the analysis for flooding loads would be included in the structural analysis report

However, the applicant's response is incomplete. The staff requests that the applicant provide more complete design specification information against flood loads, including:

(a) all materials used in design (not only concrete);

(b) a complete description of load combinations, load parameters and acceptance criteria;

(c) the safety factors for stability (sliding, overturning) and soil parameters; and

(d) the design procedures and ITAAC tables.

The staff needs this information to be able to conclude that SSC with interaction potential are designed and built to withstand the design basis flood without compromising the safety functions of the Seismic Category I SSCs.

RESPONSE:

Regarding the Item (a) above, STPNOC submitted a revised response to RAI 03.4.02-8 (see letter U7-C-STP-NRC-100193 dated August 19, 2010) that explicitly identified the flood loads to be used in the design of non-Seismic Category I structures rather than referencing the response to RAI 03.04.02-9. However, STPNOC's response to RAI 03.04.02-11, submitted concurrently with this response, provides revised flood loads to be used in the design of Seismic Category I structures. Hydrodynamic and hydrostatic flood forces used for the design of non-Seismic Category I structures will be consistent with those used in the design of Seismic Category I structures as provided in response to RAI 03.04.02-11. The response to RAI 03.04.02-8 will be revised accordingly.

Specific design specification information is provided below.

(a) all materials used in design (not only concrete)

The non-Seismic Category I structures with potential for interaction are designed to prevent structural failure resulting in collapse and potential damage to Seismic Category I Structures, Systems, and Components (SSCs). Non-Seismic Category I buildings with interaction potential do not rely on materials other than concrete or steel to prevent damage to Seismic Category I SSCs as a result of external flooding. These structures are not, however, specifically designed to preclude the entry of flood water. Flood protection to adjoining Seismic Category I buildings is afforded via flood proof doors and penetration seals in the Seismic Category I structures as discussed in COLA Part 2, Tier 2, Section 3.4.

As discussed further in response to Item (d) below, non-Seismic Category I structures with potential for interaction with Seismic Category I structures are conservatively treated as leak proof in stability calculations to maximize loads on the structure due to flood waters.

<u>(b) a complete description of load combinations, load parameters and acceptance criteria</u>

For concrete structures, the pertinent load combinations for extreme environmental loads are provided in ACI 349-97 as discussed in STPNOC's response to RAI 3.04.02-8, Revision 1.

For steel structures, combinations for extreme loads and definitions of types of loads are provided in AISC N690-94 (AISC N690-84 for standard plant structures). The combination for tornadoes, as an example, is repeated herein and will be modified as discussed below.

$1.6S \quad = D + L + R_a + T_a + W_t$

Where:

1.6S = the allowable (working) stress limit under the application of the respective types of loads

D = Dead load

L = Applicable live load

R_a = Pipe and equipment reactions generated by the postulated accident, including R_o (if applicable)

T_a = Thermal loads generated by the postulated accident, including T_o (if applicable)

W_t = Loads generated by the specified design tornado including tornado wind pressure, tornado created differential pressure, and tornado-generated missiles

It is recognized that AISC does not specifically address flood loads. In order to address the MCR breach, which is treated as an extreme load, a substitution in the above equation is appropriate. Therefore, W_t in the above equation will be replaced by F_a where F_a = extreme flood load (as described in response to RAI 03.04.02-11).

As discussed above it is not the aim of the design of II/I structures to prevent detachment of siding or failure of other features that are non-essential to global building stability. Other structural load combinations, load parameters and acceptance criteria are not applicable since materials other than concrete and steel are not relied upon in the design of non-Seismic Category I structures to prevent damage to Seismic Category I SSCs due to flooding.

<u>(c) the safety factors for stability (sliding, overturning) and soil parameters</u>

For non-Seismic Category I structures with the potential for interaction with Seismic Category I structures, the minimum safety factors for sliding and overturning are 1.1 for any extreme environmental load. Unstabilizing effects due to loads from an SSE or tornado far exceed those caused by the design basis flood load. Nonetheless, design calculations for non-Seismic Category I structures with potential for interaction with Seismic Category I SSC will document that the loads due to design basis flood are not controlling and that the minimum factors of safety are not exceeded. Resistance to unstabilizing effects is provided by dead load, soil friction, and passive soil pressure. The non-Seismic Category I structures with potential for interaction with Seismic Category I structures will be founded on backfill material. The structural analysis reports for these structures will also address the as-placed engineering properties of the backfill material and confirm that these engineering properties meet the values used in the site-specific design analyses.

(d) the design procedures and ITAAC tables

Design Procedures

As discussed in response to Item (a) above, non-Seismic Category I structures are conservatively treated as leak proof such that external flood waters can rise to the level of the design basis flood with no counteracting water pressure on the wall interiors. This is unlikely because fenestrations including doors and windows are not intended to resist flooding and cladding is not required to remain intact. For flood stability however, the buildings are treated as if the flood level rises only on the exterior of the building with no counteracting pressure from interior water, which is conservative from a stability and strength view.

SSE loading, tornado loads, and flood loads are considered in assessing global stability of non-Seismic Category I structures. In evaluating global stability, the effects of tornado loads and flood loads are considered to act on a building that remains enclosed and any relief of the unstabilizing effects of wind or flood due to in-leakage is ignored.

ITAAC Tables

This RAI notes NRC staff's agreement with STPNOC's proposal described in response to RAI 03.04.02-8, Revision 1, that, for non-Seismic Category I structures with potential for interaction, evidence of the analysis for flooding loads would be included in the structural analysis reports for these structures. As discussed in STPNOC's response to RAI 03.04.02-4 (see letter U7-C-STP-NRC-090161, October 7, 2009), Non-Seismic Category I structures with potential for interaction with Category I structures are the Turbine Building, Radwaste Building, Service Building, and Control Building Annex. Tier I DCD ITAAC for the Turbine Building (Table 2.15.11, Item 2) and the Radwaste Building (Table 2.15.13, Item 3) include provisions for a structural analysis report that concludes that under seismic conditions corresponding to an SSE, the as-built structures do not damage safety-related functions. As discussed in STPNOC's response to RAI 03.04.02-8 and agreed to by the NRC staff in this RAI, STPNOC will include evidence of the analysis of flood loads in these structural reports.

The Tier 1 DCD ITAAC for the Service Building (Table 2.15.14) does not address a structural analysis report and the Control Building Annex is a site specific non-Seismic Category I structure that does not have an associated ITAAC (COLA Part 7, Section 3.0, STD 1.2-1, Control Building Annex). Nonetheless, STPNOC will produce structural analysis reports for these structures similar to those for the Turbine Building and Radwaste Building that include evidence of the analysis of flood loads.

STPNOC does not believe that revised or additional ITAAC are warranted to address this issue. The NRC has previously considered the need for ITAAC to verify that the failure of non-Seismic Category I structures will not damage nearby Seismic Category I structures. This is discussed in the NRC's Final Safety Evaluation Report Related to the Certification of the Advanced Boiling Water Reactor, NUREG-1503 (p. 14-39):

> *"For non-seismic Category I SSCs, the need for ITAAC to verify that their failure will not impair the ability of nearby safety-related SSCs to perform their safety-related functions was assessed. Because the design detail and as-built and as-procured information for many non-safety-related systems (e.g., field-run piping and balance-of-plant systems) are not required for design certification and the spatial relationship between such systems and seismic Category I SSCs cannot be established until after the as-built design information is available, the non-seismic to seismic (II/I) interaction cannot be evaluated until the plant has been constructed. Accordingly, the design criteria for ensuring acceptable II/I interactions and a commitment for the COL applicant to describe the process for completion of the design of balance-of-plant and non-safety related systems to minimize II/I interactions and proposed procedures for an inspection of the as-built plant for II/I interactions have been specified as a COL action item in the SSAR."*

The design procedures described in this RAI response are sufficient to ensure that the non-Seismic Category I structures with potential interaction with Seismic Category I structures are adequately designed and constructed.

No COLA revision is required as a result of this RAI response.

RAI 03.04.02-11

QUESTION:

With STP letter U7-C-STP_NRC-100165, dated July 12, 2010, Attachment 1, the applicant responded to **RAI 03.04.02-9,** stating that:

"Waves generated based on the provisions of the reference given in Standard Review Plan (SRP) Section 3.4.2.11(3) are discussed in FSAR Section 2.4S.3.6, which refers to FSAR Section 2.4S.4.3.1,which concludes that the maximum flood level, including the maximum wave run-up, would be El. 34.4 ft MSL. Table 2.4S.4-8 presents the water levels due to dam break, wind set-up and wave run-up at STP 3 & 4 for the critical fetch. The dynamic load effects due to wave run-up splash of 0.4 ft above plant grade level would be negligible in comparison to out-of-plane design basis loads such as tornado wind pressure for seismic Category I structures. The methodology given by the Coastal Engineering Manual (CEM), Reference 2.4S.4-13, was adopted to estimate the wave height and wave run-up at STP 3 & 4 power block. The procedures outlined in the CEM use the wind speed, wind duration, water depth, and over-water fetch distance, and the run-up slope surface characteristics as input. Reference 2.4S.4-13 is the "Coastal Engineering Manual," U.S. Army Corps of Engineers, June 2006, which is a later version of the reference given in SRP Section 3.4.11 (3). As discussed in COLA Section 2.4S.4.2.2.4.3 and in response to RAI 03.04.02-1, the 44 pounds per square foot hydrodynamic drag force is due to velocity of the Main Cooling Reservoir breach flood flow."

During its evaluation the staff noted that the applicant's response refers to the wave action associated with the postulated river dam breaks located upstream of the Units 3 & 4-site. These events are calculated to result in a maximum flood elevation (including wave action) of 34.4ft MSL, thus only 0.40ft above nominal finished plant grade set at 34.0 ft MSL. The staff agrees that the resulting hydrodynamic and wave loads from those events are not significant. The governing flood event is however the assumed breach of the Main Cooling Reservoir which leads to a calculated flood elevation of 38.8ft MSL or nominal DBFL of 40.0ft MSL. As stated in its response, the fluid analysis has determined a flow velocity of 4.72 fps with an associated hydrodynamic surcharge fluid pressure of 44 psf. For DBFL above finished grade, SRP Section 3.4.2.II(3) requires consideration of wave load effects in the design of Seismic Category I SSC.

In its response the applicant has not evaluated the effect of water waves that may propagate on the water surface of the governing flood event. In its response to RAI 03.04.02-1 (RAI 3322 Question 13161), the applicant also referred to responses to four other RAIs (RAI 03.08.01-4, RAI 03.04.02-2, RAI 03.04.02-4, and RAI 03.04.02-5) for the resolution of RAI 03.04.02-1. The applicant is therefore requested to evaluate the effect of water waves that may propagate on the water surface of the governing flood event, and to track the closure status of the above noted four RAIs. The staff needs this information in order to be able to conclude that the above defined DBF effects are adequately accounted for in the design of Seismic Category I SSC pursuant to SRP Section 3.4.2.II(3).

RESPONSE:

Coincidental hydrodynamic wind wave forces were not considered with the conservative Main Cooling Reservoir (MCR) breach flood level because of the short duration of this flood. In addition, the relevant NRC and industry guidance provide for the consideration of wind-generated waves for design flood level and their effects on safety-related structures only for potential flooding due to hydrologic causes, such as Probable Maximum Precipitation, and does not provide for consideration of wind-generated waves coincident with a non-hydrologic failure, such as the postulated breach of the MCR dike breach.

To respond to this RAI, however, a 2-year fastest mile wind speed of 50 mph, based on COLA Reference 2.4S.4-7, is conservatively applied coincident with the MCR breach flood level to determine the hydrodynamic load due to the wind generated waves. The methodology given in the Coastal Engineering Manual (CEM), COLA Reference 2.4S.4-13 is used to estimate the wave height and wave forces on the vertical walls of the STP 3 & 4 power block buildings.

1. Hydrodynamic Wind Wave Forces on the Safety-Related Structures:

Based on the site layout and considering the sheltering effect of other buildings or structures on the site, the controlling fetch length will be due to the westerly winds. Therefore, the longest fetch on the west facing Unit 4 safety-related structures is determined. For this governing condition, the wave height is calculated for the above wind speed, fetch and the depth of water along the fetch. Based on this, a significant non-breaking wave with a wave height (H_s) of 1.25 feet and a period (T) of 1.7 seconds would be generated. Considering a 1% wave height (H_1= 1.67 H_s) of 2.1 feet, per COLA Reference 2.4S.4-7, the wave force due to the wind generated waves is calculated and conservatively applied to all the safety-related structures including those for Unit 3.

The resultant hydrodynamic wave force is calculated to be 603 pounds (0.6 kips) per foot length of the vertical wall corresponding to the maximum breach flood level of 38.8 feet. The wave force diagram is shown in Figure 3.4-1, included with the COLA mark-up at the end of this response.

As seen from Figure 3.4-1, the total hydrostatic and hydrodynamic pressure at grade elevation 34'-0" is 339 psf (i.e. considering a sediment-laden water density of 63.85 lb/ft^3, 306.5 psf + 32.5 psf = 339 psf). This pressure is less than the hydrostatic pressure due to conservatively established design basis flood level of 40'-0" (i.e. 6 x 62.4 = 374 psf). Therefore, inclusion of wind generated wave forces does not affect design of below grade walls.

2. Maximum Water Level due to Wind-Generated Waves near the Safety-Related Structures:

Due to the waves generated by the postulated wind, the water level near the safety-related structures will fluctuate above and below the still water level caused by the MCR dike breach flood. As stated in Item 1 above, the water levels near the Unit 4 safety-related structures are affected more than the water levels near the Unit 3 structures due to the controlling westerly winds. Therefore, the rise in water level due to wind wave effect near Unit 4 safety-related structures is considered as the upper bound water level fluctuation for the Unit 3 structures also.

Following are the maximum water levels near Unit 4 safety-related structures due to MCR dike breach flood and the fluctuation of the water level due to the wind waves:

- Maximum water level due to MCR breach flood near the Unit-4 Ultimate Heat Sink (UHS) = 38.8 feet
- Maximum water level due to MCR breach flood near the Unit-4 power block structures = 38.2 feet
- Maximum periodic rise in water level due to wind wave action = 3.1 feet (see Figure 3.4-1)

Including the fluctuation in water level due to wind wave effect:

- The maximum water level near the Unit-4 UHS = 38.8 + 3.1 = 41.9 feet
- The maximum water level near the Unit-4 power block structures = 38.2 + 3.1 = 41.3 feet

The UHS and Reactor Service Water (RSW) Pump Houses are designed to be watertight below 50 feet MSL. All the power block safety-related structures are watertight below elevation 41.0 feet MSL due to one foot threshold provided above the design basis flood level of 40 feet MSL. Any periodic splash flooding above the 41-foot elevation up to the wave run-up elevation of 41.3 feet MSL will be minor and would be taken care of with normal housekeeping and will not affect the safety-related function of the structures.

Consistent with Standard Review Plan Section 3.4.2 requirements, and considering the above, the following criteria will be applied for the design of the safety-related structures:

a) Flotation stability evaluations shall be based on the buoyancy calculations using the conservatively established design basis flood level of 40’-0” MSL.

b) The lateral loads on the structural walls and overturning moment on the structure will include the effect of the wave-generated hydrodynamic forces, as discussed in Item 1 above and floating debris (see response to RAI 03.08.01-10 which is being submitted concurrently with this response). As such, external walls of the structures shall be capable of resisting the following loads:

    - Hydrostatic force considering a conservatively established design basis flood level of 40’-0” MSL.
    - Hydrodynamic drag force of 44 psf due to flood water flow, applicable to above grade portion.
    - Wind generated wave forces as shown in Figure 3.4-1, applicable to above grade portion.
    - Impact due to a 500 lbs floating debris traveling at 4.72 ft/sec.

c) Watertight seals protecting the exterior penetrations and seismic gaps against flooding shall be designed to take into account the increase in hydrostatic head due to the design basis flood elevation of 40'-0" MSL.

Application of the above criteria will impact previously submitted responses to the following RAIs and COLA sections:

- COLA Section 2.4S
- COLA Section 3.4
- RAI 03.04.02-1
- RAI 03.04.02-2
- RAI 03.04.02-4
- RAI 03.04.02-5
- RAI 03.08.01-4
- RAI 03.08.01-7
- RAI 03.08.04-18
- RAI 03.08.04-22

Impact on COLA Sections 2.4S and 3.4:

See COLA changes provided at the end of this response.

Impact on RAI 03.04.02-1:

See revised response RAI 03.04.02-1 Revision 1 being provided concurrently with this response.

Impact on RAI 03.04.02-2:

See response to follow-up RAI 03.04.02-6 Revision 1 being provided concurrently with this response.

Impact on RAI 03.04.02-4:

See response to RAI 03.04.02-10 being provided concurrently with this response.

Impact on RAI 03.04.02-5:

As noted in the criteria provided under Item 2C above, the seals protecting the exterior penetrations and seismic gaps against flooding shall be designed to take into account the increase in hydrostatic head due to the design basis flood elevation of 40'-0" MSL. This criterion is same as that previously used in the response to RAI 03.04.02-5. Therefore, there is no change in the response to RAI 03.04.02-5.

Impact on RAI 03.08.01-4

See revised response RAI 03.08.01-4 Revision 1 being provided concurrently with this response.

Impact on RAI 03.08.01-7

See revised response RAI 03.08.01-7 Revision 1 being provided concurrently with this response.

Impact on RAI 03.08.04-18

The flood loading including the hydrodynamic forces due to flood water flow and wind generated waves is bounded by the seismic loading considered in the design of the Radwaste Building. The exterior, above grade walls of the Radwaste Building are 3 ft thick, spanning nearly 60 ft (i.e. from elevation 35 ft to roof elevation of approximately 95 ft) which have been qualified/designed for seismic II/I requirement, considering an earthquake input that envelops 0.3g Regulatory Guide 1.60 response spectrum and the induced acceleration response spectrum due to site-specific Safe Shutdown Earthquake.

For COLA changes due to this response, see supplement 1 for RAI 03.08.04-18 Revision 1 being provided concurrently with this response.

Impact on RAI 03.08.04-22

The design of the RSW Piping Tunnels and UHS/RSW Pump House for flood loading, including the hydrodynamic forces due to flood water flow and wind generated waves, is bounded by the existing design for the following reasons:

- The only portions of the RSW Piping Tunnels which are located above grade are the access shafts which have 3 ft thick walls with minimum reinforcement of #10 at 12 inch spacing (except where wall reinforcement is #8 at 12 inch spacing for a 4 ft clear span). The maximum span for these access shaft walls is 30 ft. Assuming a maximum uniform load of 0.4 k/ft (which exceeds the maximum flood load at grade level) and a maximum span of 30 ft, the maximum induced shear and moment will be 6 k/ft and 45 k-ft/ft, respectively. The shear and moment capacity of a 3 ft thick wall with #10 bars at 12 inch spacing will far exceed the shear and moment due to these loads.

- Exterior walls of the UHS/RSW Pump House subject to flooding loads are 6-foot-thick reinforced concrete walls with a minimum reinforcement of #11 at 12 inch spacing. Design of these walls is governed by loadings other than flood loading because the induced shears and moments due to flood loading will be far less than the minimum shear and moment capacity of these walls. It should also be noted that the flood forces acting on the 6-foot-thick UHS basin exterior walls will oppose the hydrostatic load due to water within the basin.

For COLA changes due to this response, see RAI 03.08.04-22 Revision 1 being provided concurrently with this response.

The STP Units 3 and 4 COLA will be revised as follows as a result of this response.

2.4S.4.2.2.4.3 Hydrodynamic Forces

The maximum water levels and velocities obtained near Units 3 and 4 were used to assess the hydrodynamic loadings on the plant buildings. Figures 2.4S.4-21(g) and 2.4S.4-21(h) show the time-dependent plots of the velocities at this location during the east and west breach scenarios, respectively. The peak velocities observed were 4.72 and 4.68 feet per second for the east and west breach scenarios, respectively. Figures 2.4S.4-21(g) and 2.4S.4-21(h) also show the sediment concentrations predicted by the SED2D model. The sediment-laden water density was used for hydrodynamic load calculations. The figures show that the sediment concentrations at the time and location of peak velocities would be 16.5 kg/m3 and 15 kg/m3 for the east and west breach scenarios, respectively. However, Figure 2.4S.4-21(g) shows a maximum concentration of 23 kg/m3 occurring at approximately T = 1.3 hours. Conservatively, the maximum sediment concentration was used in conjunction with the maximum velocity to determine the hydrodynamic loads on the STP 3 and 4 plant facilities. Selecting a 23 kg/m3 sediment concentration, a water density of 1023 kg/m3 or 63.85 lb/ft3 was used for load calculations. The maximum hydrostatic force on any plant building would be due to the depth of floodwater at the maximum water level. Hydrodynamic loads were calculated using the drag force formula with a drag coefficient conservatively set to 2.0, as presented below:

Force (lb/ft2) = 2.0 x Density (lb/ft3) x Velocity2 (ft2/sec2) / 2g

The maximum drag force due to the maximum velocity of flow near the plant buildings is estimated as 44 pounds per square foot of the projected submerged area of the buildings.

The hydrodynamic loads due to wind-generated waves have also been calculated. A two year fastest mile wind speed of 50 mph, based on Reference 2.4S.4-7, is conservatively applied coincident with the Main Cooling Reservoir (MCR) breach flood level. The methodology given in the Coastal Engineering Manual (CEM), Reference 2.4S.4-13, is used to estimate the wave height and wave forces on the vertical walls of the power block buildings.

Based on the site layout and considering the sheltering effect of other buildings or structures on the site, the controlling fetch length will be due to the westerly winds. Therefore, the longest fetch on the west facing Unit 4 safety-related structures is determined. For this governing condition, the wave height is calculated for the above wind speed, fetch and the depth of water along the fetch. Based on this, a significant non-breaking wave with a wave height (H_s) of 1.25 feet and a period (T) of 1.7 seconds would be generated. Considering a 1% wave height (H_1 = 1.67 H_s) of 2.1 feet, per Reference 2.4S.4-7, the wave force due to the wind generated waves is calculated and conservatively applied to all the safety-related structures including those for Unit 3.

The resultant hydrodynamic wave force is calculated to be 603 pounds (0.6 kips) per foot length of the vertical wall corresponding to the maximum breach flood level of 38.8 feet. The wave force diagram is shown in Figure 3.4-1.

Due to the waves generated by the postulated wind the water level near the safety-related structures will fluctuate above and below the still water level caused by the MCR dike breach flood. As stated above, the water levels near the Unit 4 safety-related structures are affected more than the water levels near the Unit 3 structures due to the controlling westerly winds. Therefore, the rise in water level due to wind wave effect near Unit 4 safety-related structures is considered as the upper bound water level fluctuation for the Unit 3 structures also.

Following are the maximum water levels near Unit 4 safety-related structures due to MCR dike breach flood and the fluctuation of the water level due to the wind waves. The MCR dike breach flood levels are described in Section 2.4S.4.

- Maximum water level due to MCR breach flood near the Unit-4 Ultimate Heat Sink (UHS) = 38.8 feet
- Maximum water level due to MCR breach flood near the Unit-4 power block structures = 38.2 feet.
- Maximum periodic rise in water level due to wind wave action = 3.1 feet (see Figure 3.4-1)

Including the fluctuation in water level due to wind wave effect;

- The maximum water level near the Unit-4 UHS = 38.8 + 3.1 = 41.9 feet.
- The maximum water level near the Unit-4 power block structures = 38.2 + 3.1 = 41.3 feet.

The UHS and Reactor Service Water (RSW) Pump Houses are designed to be watertight below 50 feet MSL. All the power block safety-related structures are watertight below elevation 41.0 feet MSL due to one foot threshold provided above the design basis flood level of 40 feet MSL. Any periodic splash flooding above the 41-foot elevation up to the wave run-up elevation of 41.3 feet MSL will be minor and would be taken care of with normal housekeeping and will not affect the safety-related function of the structures.

3.4.2 Analytical and Test Procedures

STP DEP T1 5.0-1

Since the design basis flood elevation is at El. 40.0 ft (see Subsection 2.4S.2.2), 182.9 cm above the finished plant grade, the lateral hydrostatic and hydrodynamic pressure on the structures due to the design flood water level, as well as ground and soil pressures, are calculated.

As discussed in Section 2.4S.4.2.2.4.3, the hydrodynamic force due to the wind-generated wave action on building walls has been calculated as shown in Figure 3.4-1.

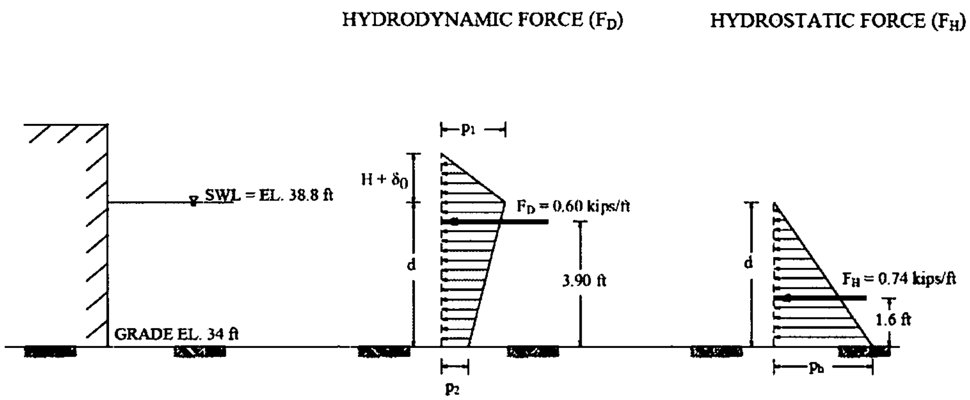
Consistent with Standard Review Plan Section 3.4.2 requirements, and the discussion provided in Section 2.4S.4.2.2.4.3, the following criteria will be applied for the design of the safety-related structures:

a) Flotation stability evaluations shall be based on the buoyancy calculations using the conservatively established design basis flood level of 40'-0" MSL.

b) The lateral loads on the structural walls and overturning moment on the structure will include the effect of the wave-generated hydrodynamic forces. As such, external walls of the structures shall be capable of resisting the following loads:

- Hydrostatic force considering a conservatively established design basis flood level of 40'-0" MSL.
- Hydrodynamic drag force of 44 psf due to flood water flow, applicable to above grade portion.
- Wind generated wave forces as shown in Figure 3.4-1, applicable to above grade portion.
- Impact due to a 500 lbs floating debris traveling at 4.72 ft/sec.

c) Watertight seals protecting the exterior penetrations and seismic gaps against flooding shall be designed to take into account the increase in hydrostatic head due to the design basis flood elevation of 40'-0" MSL.



F_D = Resultant hydrodynamic force (in kips per linear foot) equivalent to the hydrodynamic wave pressure (p_1 and p_2) acting on the structure
F_H = Resultant hydrostatic force (in kips per linear foot) equivalent to the hydrostatic pressure (p_h) acting on the structure
H = Non-Breaking Wave Height(1%) = 2.1 ft
T = Non-Breaking Wave Period = 1.7 sec
d = depth of water at the structure = 4.8 ft
δ_0 = vertical shift of the wave crest and trough at the structure = 1.0 ft

p_1 = hydrodynamic wave pressure at the still water level = 132.8 lb/ft²
p_2 = hydrodynamic wave pressure at the base of the structure = 32.5 lb/ft²

p_h = hydrostatic pressure at the base of the structure = 306.5 lb/ft²

Figure 3.4-1
Non-Breaking Wave Force on Vertical Wall

RAI 03.07.01-25

QUESTION:

Follow-up Question to RAI 03.07.01-17 (STP-NRC-100035)

10CFR50 Appendix S requires that seismic evaluation must take into account soil-structure interaction (SSI) effects. STP has performed a site-specific SSI analysis to confirm that the ABWR DCD results envelop the results of the site-specific SSI analysis of the RB and CB. Regarding this reconciliation analysis the staff needs the following additional information to determine that site-specific SSI analysis adequately predicts the RB and CB seismic response:

1. In response to Item 1b of RAI 03.07.01-17, the applicant has provided comparison of the strain compatible shear wave velocity profiles for the backfill with those of the in-situ and DCD UB1D150 soil columns in Figure 3A-230a. Based on this comparison, the applicant has concluded that a separate confirmatory SSI analysis of the RB and CB incorporating backfill is not necessary because the lower and upper bound shear wave velocities of the backfill are enveloped by those of the in-situ soils and those used in DCD. Although this assertion is acceptable for the lower bound backfill properties, it has not been shown in Figure 3A-230a that the strain compatible DCD shear wave velocity profile envelop the upper bound backfill properties where the velocities exceed those of the in-situ upper bound profile and DCD UB1D150 at depths of approximately 12 to 52 feet below grade (see Figure 3A-230a). While the UB1D150 may be the lowest shear wave velocity case in the DCD, the applicant is requested to provide in the same Figure (3A-230a) comparison of the DCD upper bound strain compatible soil case that envelops the upper bound backfill properties.

2. In response to Item 1b of RAI 03.07.01-17 with respect to the strain-compatible damping properties, the applicant has provided comparison of the soil damping profiles for the backfill with those of the in-situ soil columns in Figure 3A-230b. Based on this comparison, the applicant has concluded that the backfill damping is generally higher than those of the in-situ soils, and thus bounded by the in-situ soil properties. A review of the results presented in Figure 3A-230b shows the lower-bound damping profile for the backfill to be significantly higher than that of the in-situ soils. Because the SSE design motion is specified at the free-field ground surface, a higher damping in the backfill material may result in a higher motion at the foundation level as compared with that obtained from the in-situ soil column with lower damping to compensate for the higher attenuation of the motion in the backfill soils. As such, the applicant is requested to provide further justification that the higher damping in the backfill material for the lower bound case will not result in foundation motions that exceed those of DCD.

3. In the response to Item 2 of RAI 03.07.01-17, the applicant has stated that the Poisson's ratio has been capped at 0.48 for saturated soils in calculating the compression wave velocity. This results in calculated compression wave velocities lower than 5000 ft/sec in saturated soils when the shear wave velocities drop below approximately 980 ft/sec. For example, as shown in Tables 3H.6-1b through 3H.6-2c (see the enclosure to STP's response to RAI 03.07.02-17),

approximately 57, 75 and 240 feet of the respective soil column of the in-situ upper bound, lower bound and mean soil cases have calculated P-wave velocities less than 5000 ft/sec. The use of compression wave velocities in saturated soils less than 5000 ft/sec will not allow the higher frequency components of the vertical motion to be transmitted into the structure and may result in less conservative response. As such, the applicant is requested to assess the impact of using P-wave velocities lower than 5000 ft/sec in saturated soils on the response of the structure including in-structure response spectra by performing a sensitivity study and comparing the results for two cases: Case 1 will cap Poisson's ratio at 0.48 for saturated soils and let P-wave velocity drop below 5000 ft/sec (similar to the procedure stated by the applicant) and Case 2 will set P-wave velocity to 5000 ft/sec in saturated soils and allow Poisson's ratio to rise above 0.48 depending on the strain-compatible shear wave velocities.

RESPONSE:

The following provides the response to parts 1 and 2 of this RAI. The response to part 3 of this RAI will be provided in a supplemental response by October 25, 2010.

1. As requested, a revised COLA Figure 3A-230a is provided in which the strain-compatible shear wave velocity for DCD VP3 soil column is added. The DCD VP3 is the next higher shear wave velocity soil profile after UB soil profile. The strain-compatible soil properties for the VP3 have been obtained from the free-field analysis, using the same procedure as described in response 1b of RAI 03.07.01-17 (submitted with letter U7-C-STP-NRC-100035, dated February 4, 2010). The revised Figure 3A-230a shows that the DCD VP3 strain-compatible shear wave velocities completely envelope the upper bound backfill strain-compatible shear wave velocities.

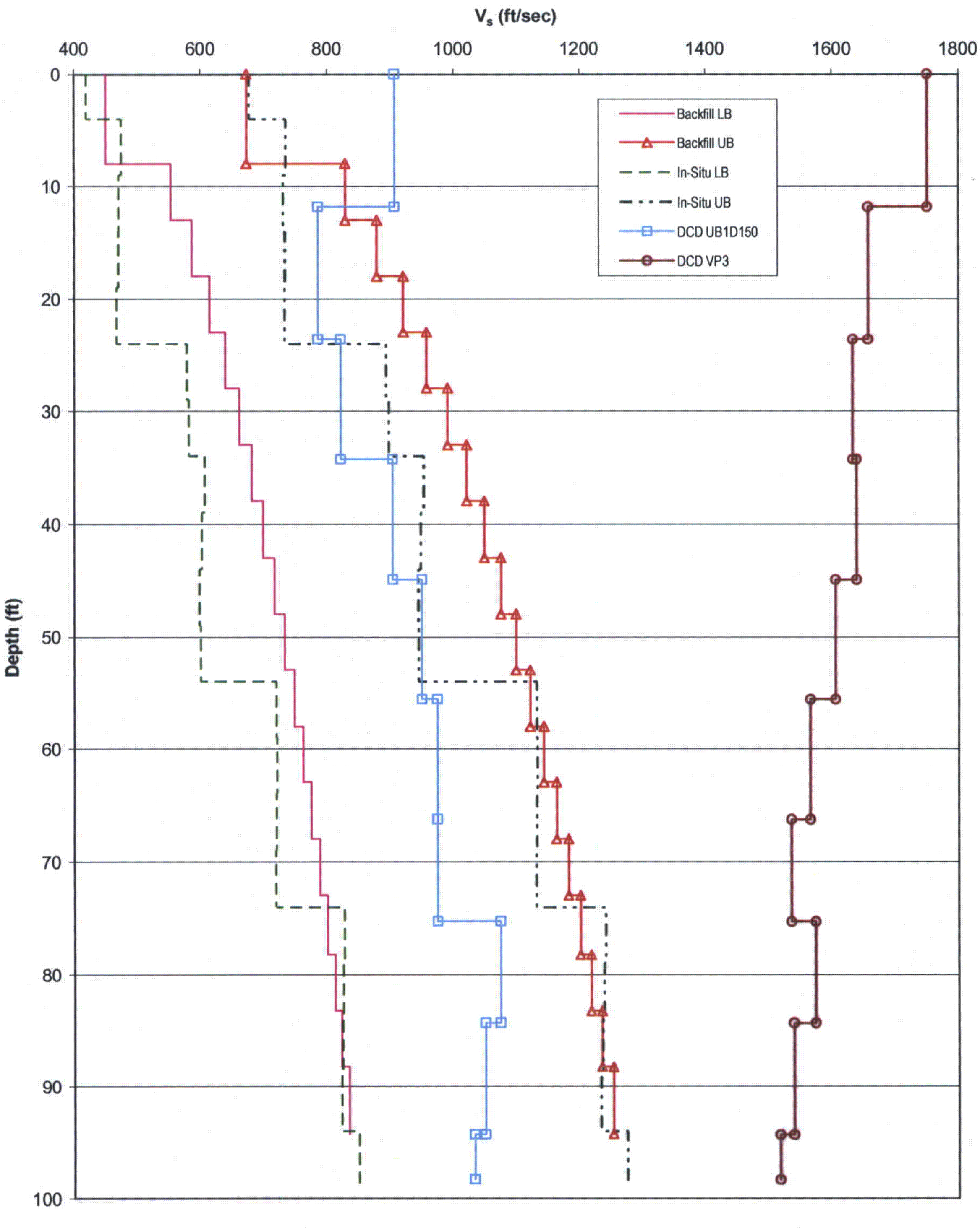
2. The estimated material damping for lower-bound (LB) backfill is 3%. The material damping for LB in-situ soil varies in the range of about 1.67% to 3.1%, in general, around 2.25%. The low damping of about 1.67% is for top 4 ft soil depth. The use of backfill material damping may result in somewhat higher motion at the foundation level as compared to the use of the in-situ soil material damping, but the difference in the foundation level motions due to difference in the two damping values (i.e. 3% for backfill and 2.25% for in-situ soil) will be very small. Furthermore, in the Soil Structure Interaction analysis, because of much higher radiation damping (generally higher than 20% for horizontal and vertical motions, as demonstrated in NUREG/CR-5956, "Consideration of Uncertainties in Soil-Structure Interaction Computations", prepared by C.J. Costantino and C.A. Miller), the small difference in the material damping will have insignificant effect on the final responses.

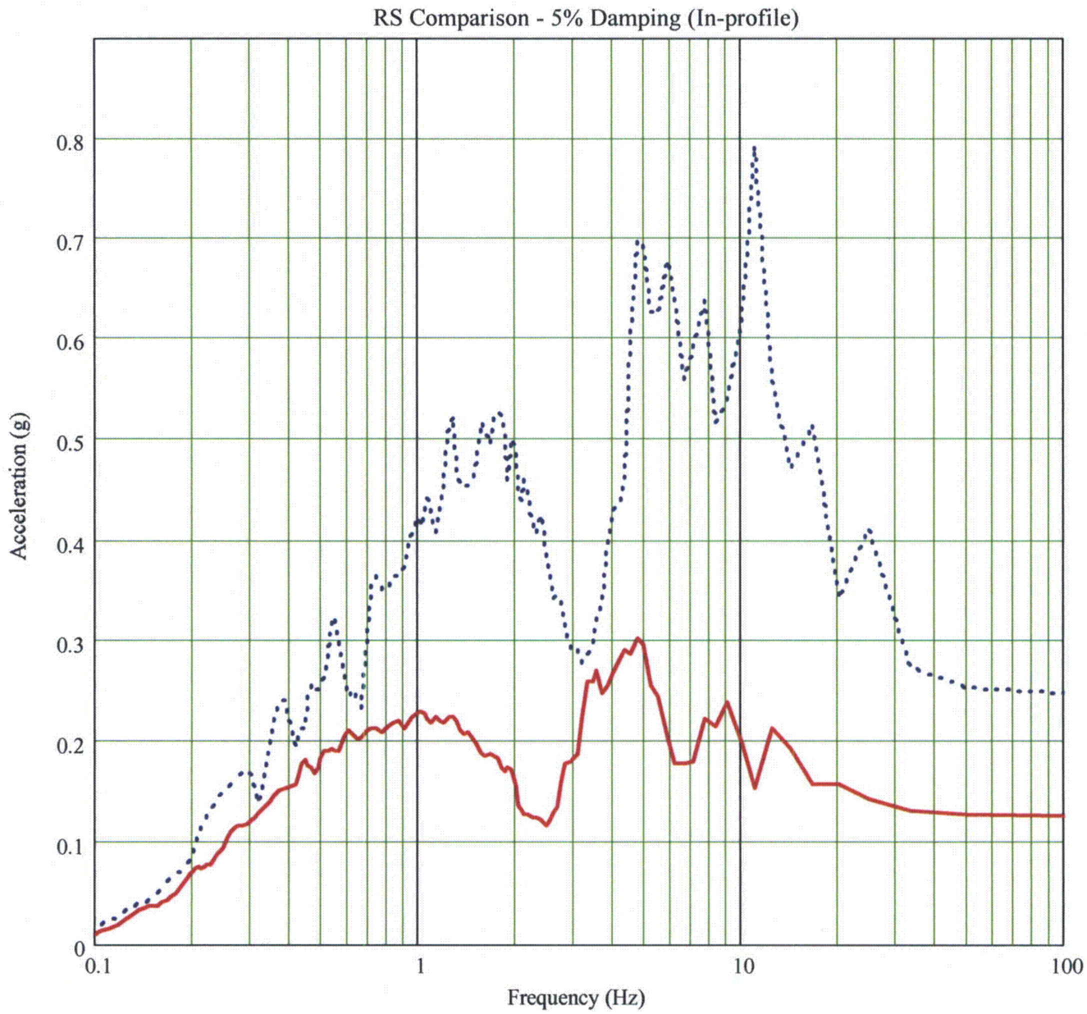
   To further demonstrate that the higher damping in the backfill material for lower bound case will not result in foundation motions that exceed those of DCD, free-field SHAKE2000 analyses have been performed for lower bound backfill profile with 3% damping (with site specific Safe Shutdown Earthquake (SSE) input motion at grade) and DCD UB strain compatible soil profile (with 0.3g R.G. 1.60 input motion at grade). The

Reactor Building (RB) and Control Building (CB) foundation levels (bottom of basemat and bottom of mudmat) response spectra obtained from the two analyses are compared. The comparisons are made for both outcrop and in-profile motions. Figures 03.07.01-25a through 03.07.01-25h show the comparisons of foundation level 5% damped spectra for the CB. Figures 03.07.01-25i through 03.07.01-25p show the comparisons of foundation level 5% damped spectra for the RB. The comparisons show that the DCD foundation motions exceed the corresponding foundation motions obtained from lower bound backfill with 3% damping.

The COLA Figure 3A-230a submitted with this response will replace COLA Figure 3A-230a submitted with the response to RAI 03.07.01-17 (submitted with STP letter U7-C-STP-NRC-100035, dated February 4, 2010).

Figure 3A-230a Comparison of Strain-Compatible Shear Wave Velocity Profiles for Backfill

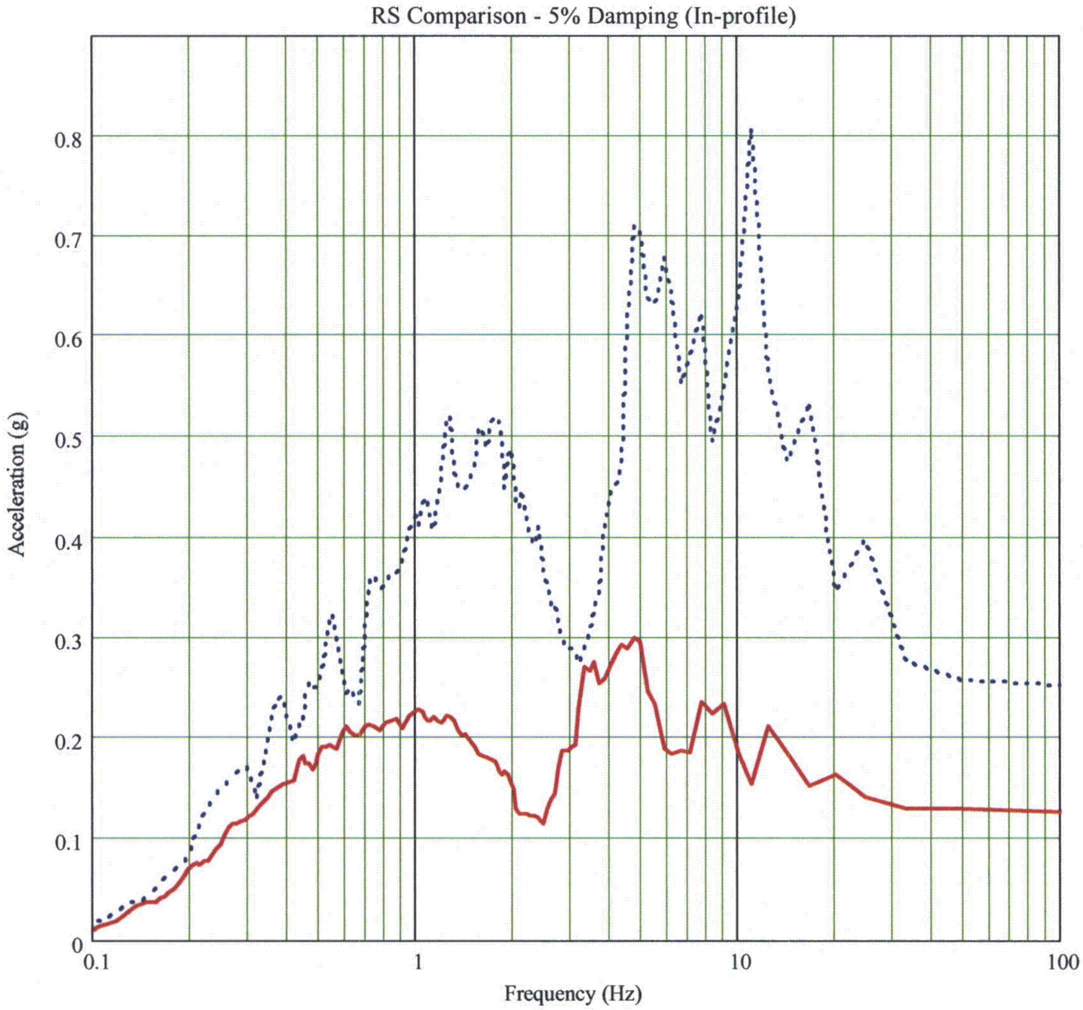




Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

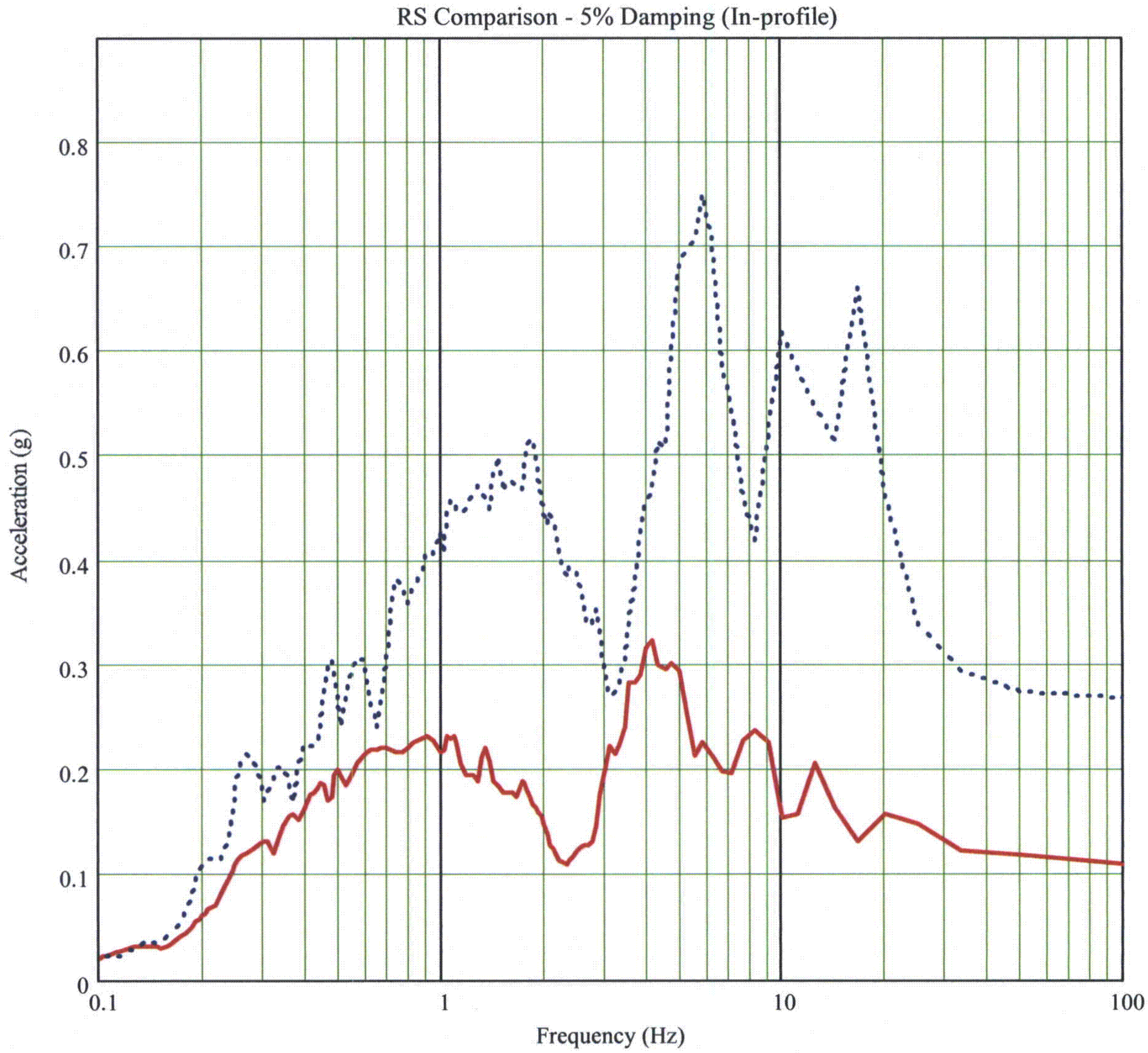
Figure 03.07.01-25a: In-Profile Spectral Comparison in NS Direction at Bottom of Control Building Foundation Mat (76.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

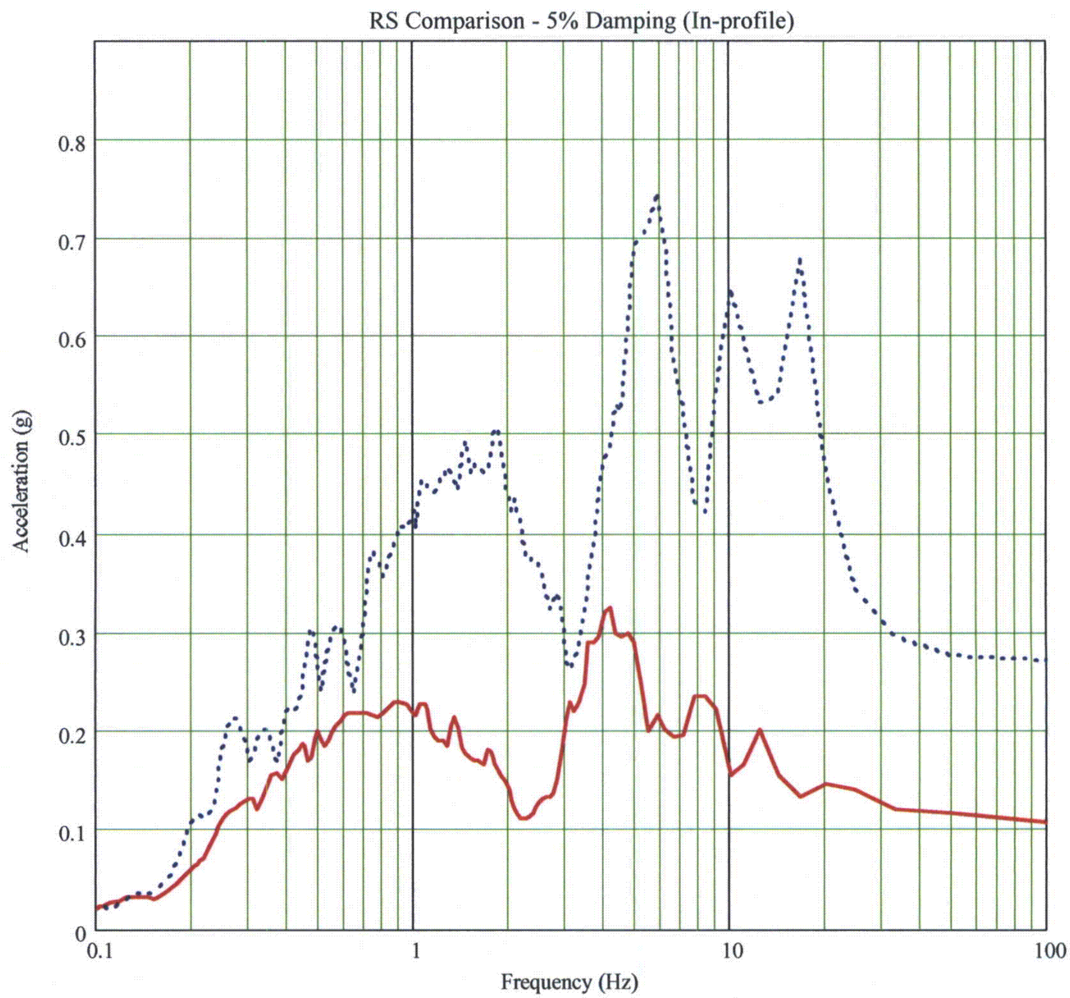
Figure 03.07.01-25b: In-Profile Spectral Comparison in NS Direction at Bottom of Control Building Mudmat (78.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

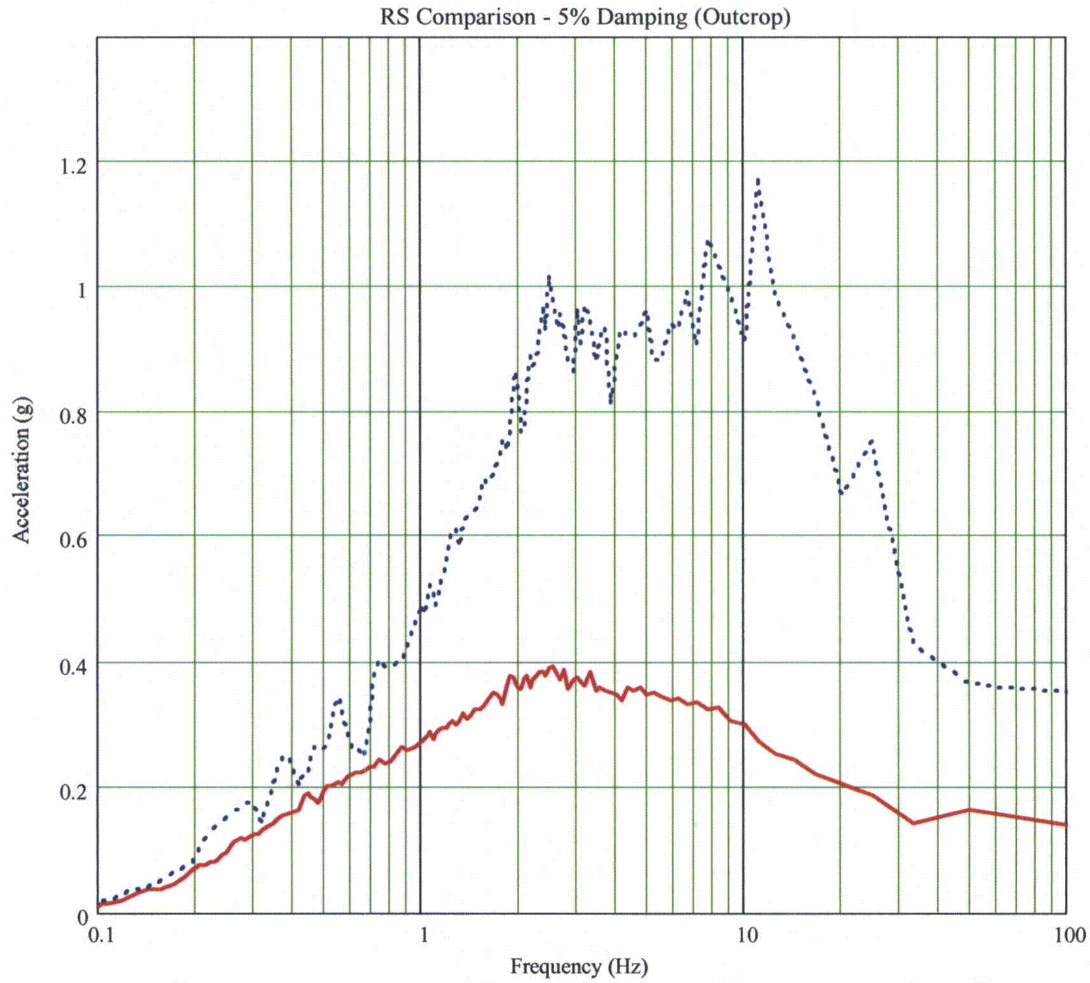
Figure 03.07.01-25c: In-Profile Spectral Comparison in EW Direction at Bottom of Control Building Foundation Mat (76.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

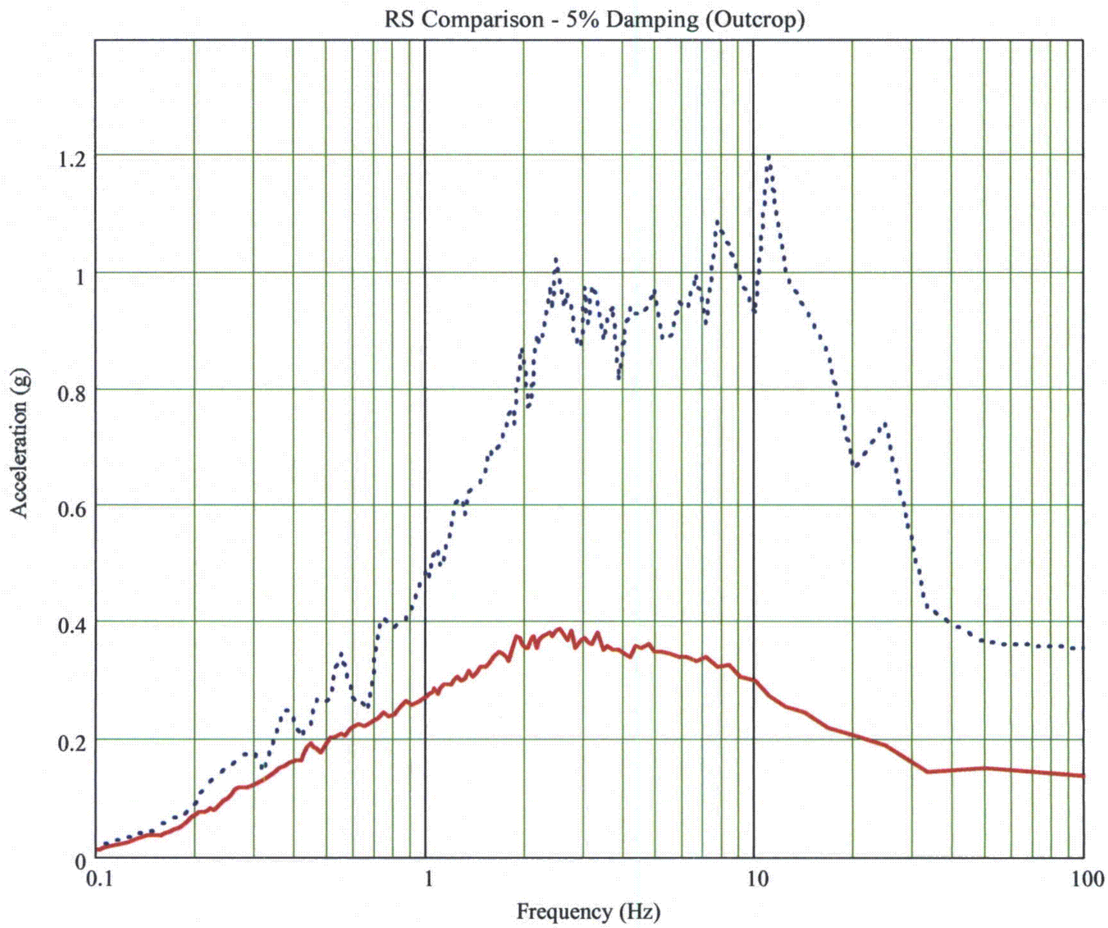
Figure 03.07.01-25d: In-Profile Spectral Comparison in EW Direction at Bottom of Control Building Mudmat (78.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

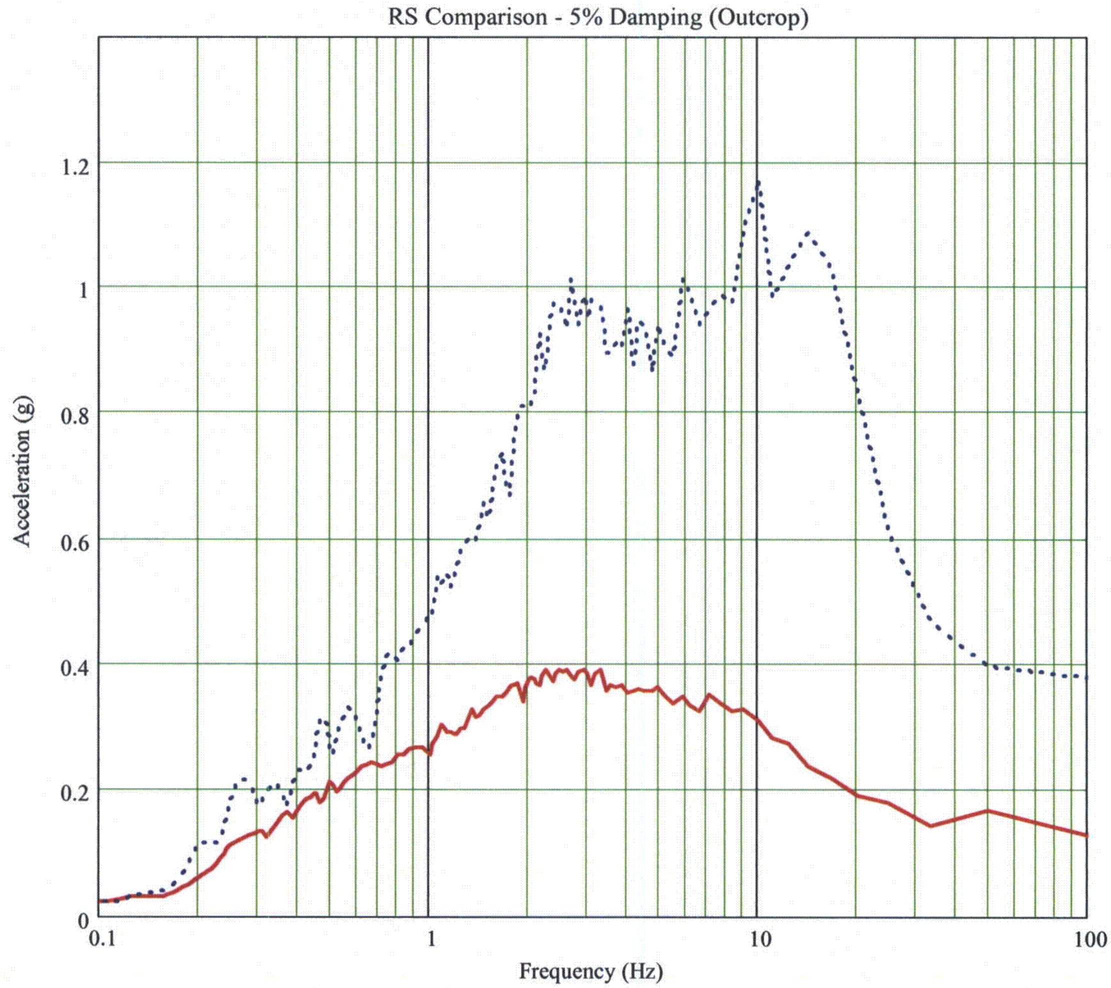
Figure 03.07.01-25e: Outcrop Spectral Comparison in NS Direction at Bottom of Control Building Foundation Mat (76.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

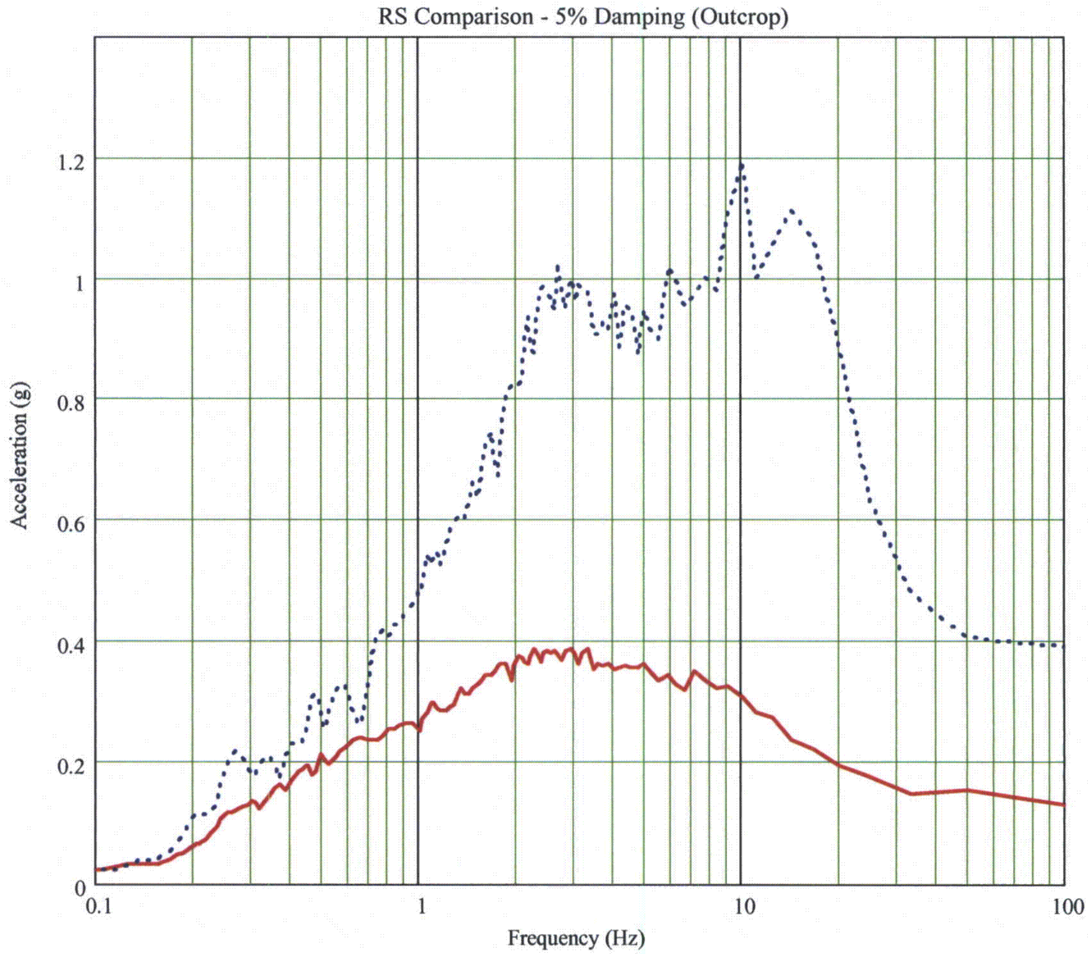
Figure 03.07.01-25f: Outcrop Spectral Comparison in NS Direction at Bottom of Control Building Mudmat (78.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

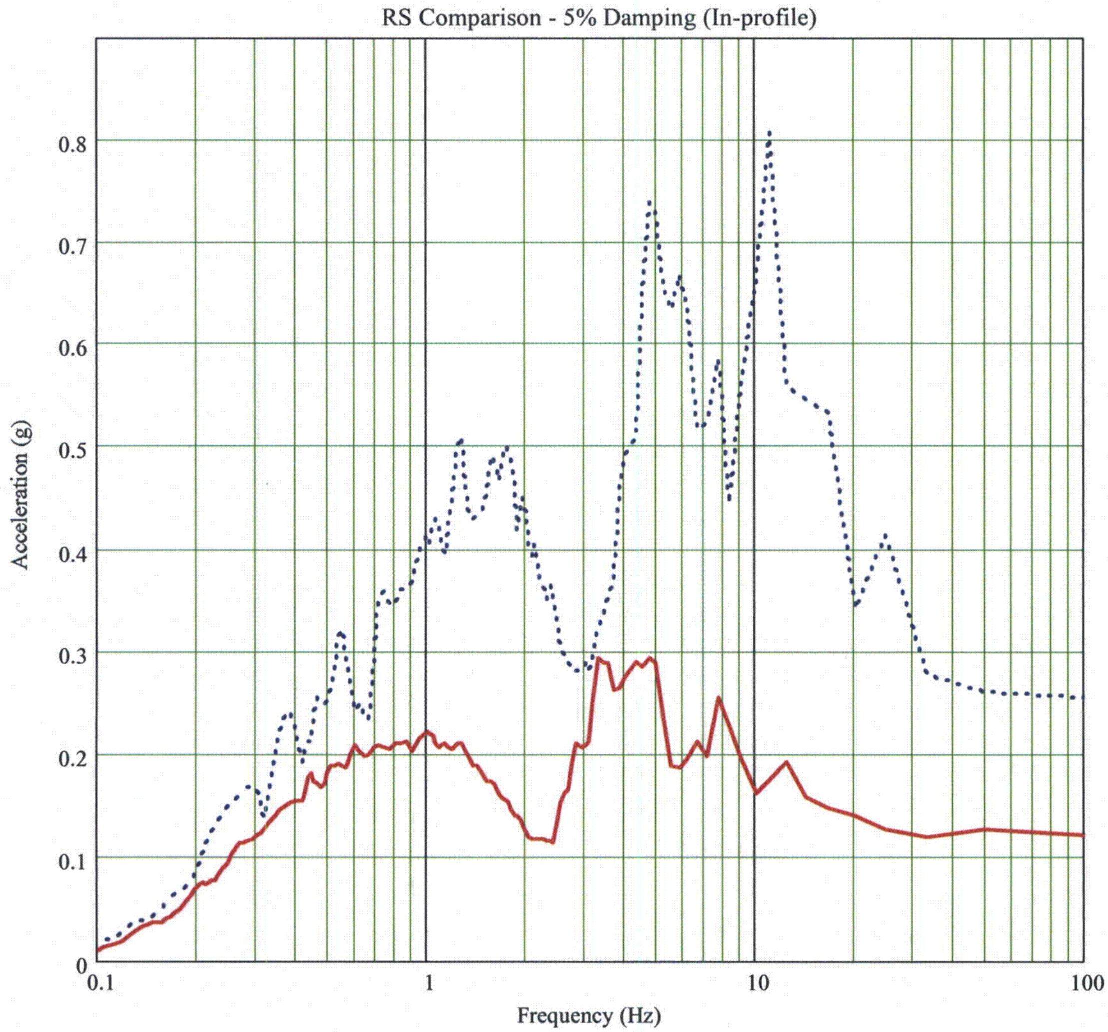
Figure 03.07.01-25g: Outcrop Spectral Comparison in EW Direction at Bottom of Control Building Foundation Mat (76.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

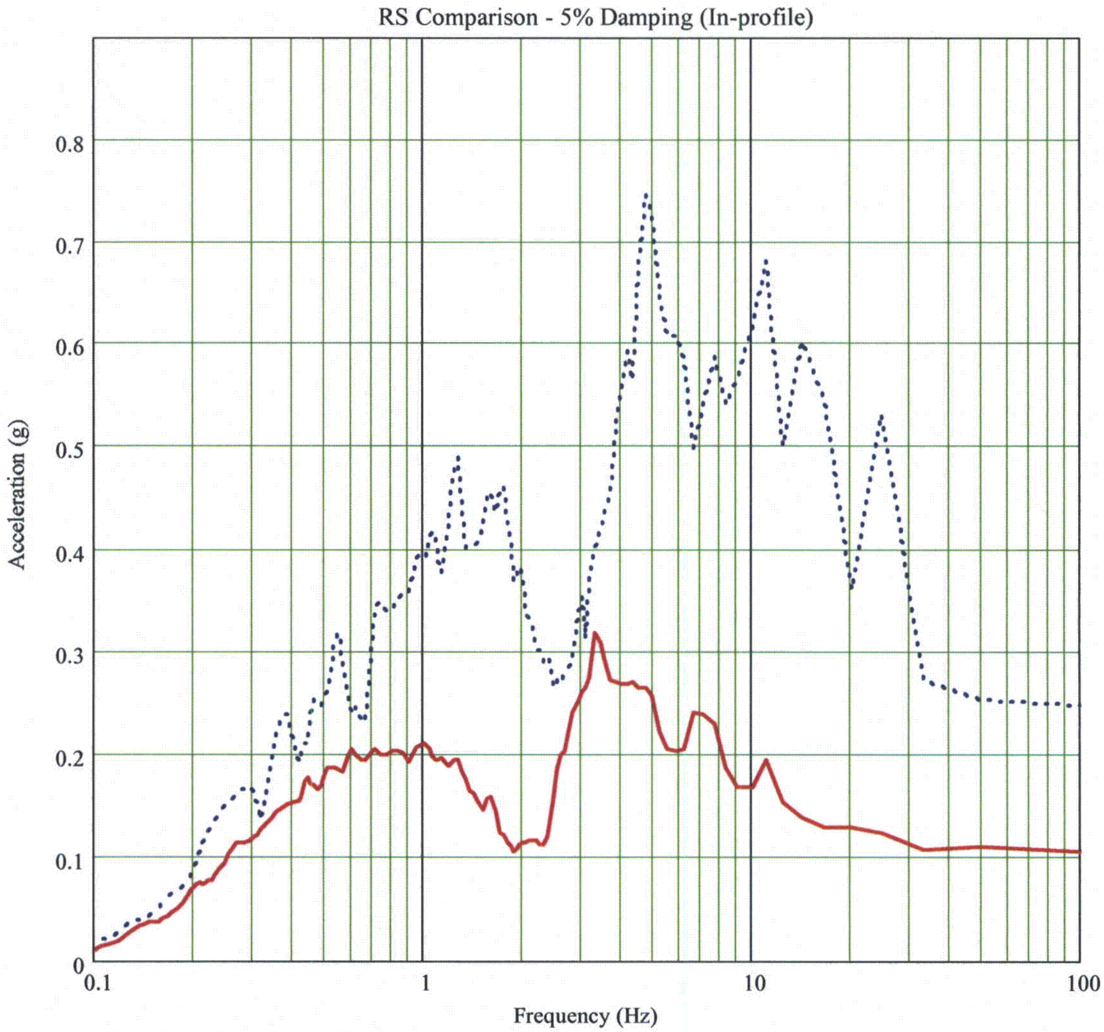
Figure 03.07.01-25h: Outcrop Spectral Comparison in EW Direction at Bottom of Control Building Mudmat (78.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

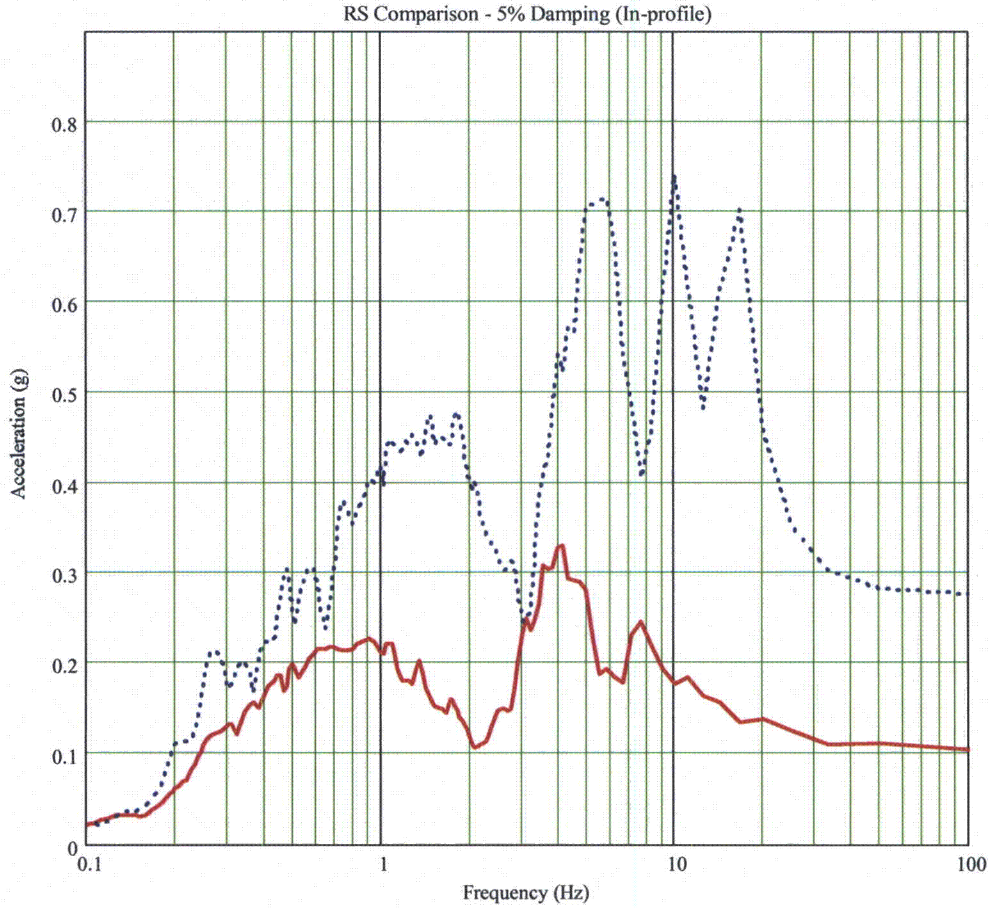
Figure 03.07.01-25i: In-Profile Spectral Comparison in NS Direction at Bottom of Reactor Building Foundation Mat (84.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

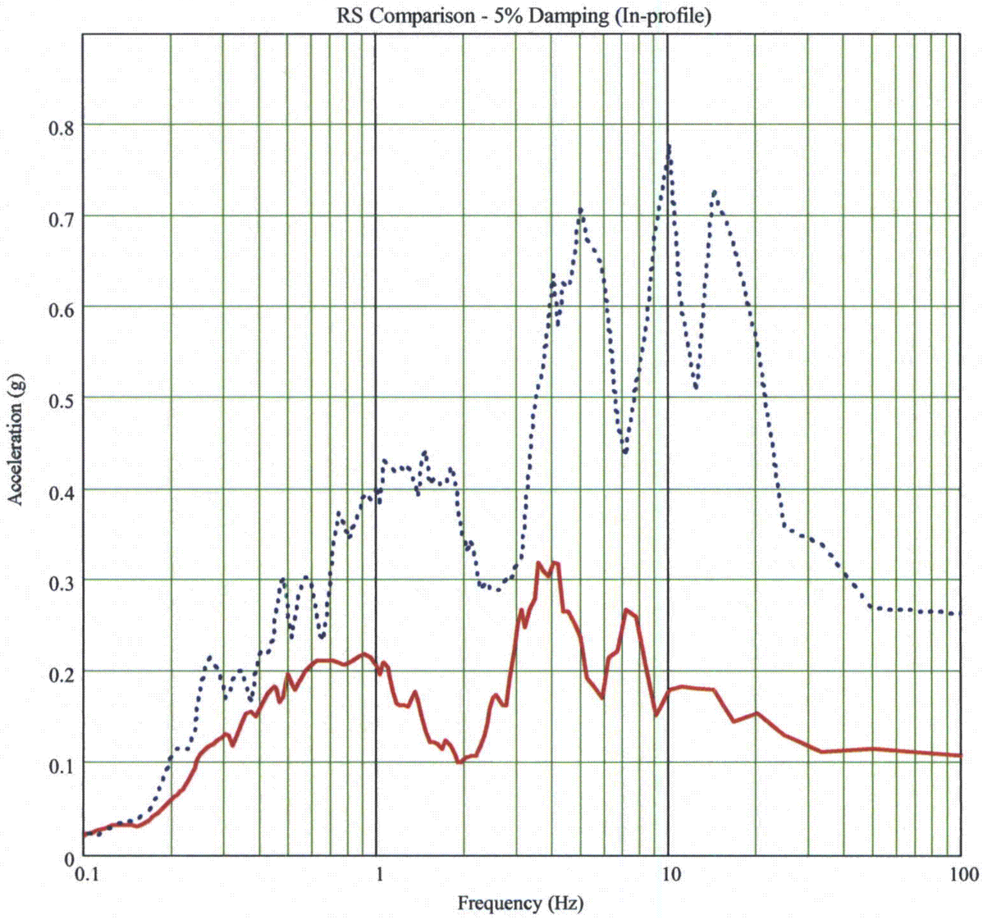
Figure 03.07.01-25j: In-Profile Spectral Comparison in NS Direction at Bottom of Reactor Building Mudmat (94.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

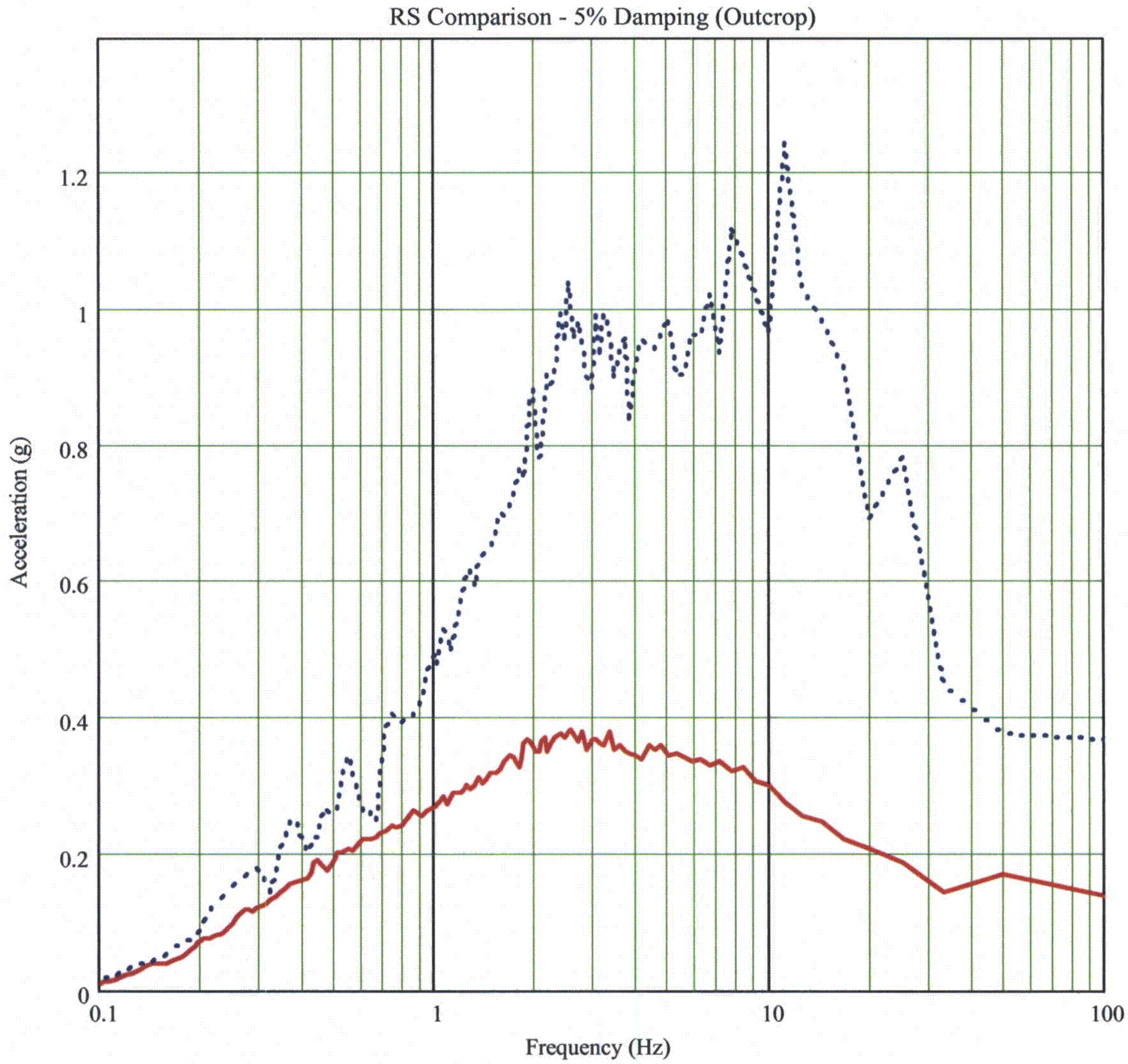
Figure 03.07.01-25k: In-Profile Spectral Comparison in EW Direction at Bottom of Reactor Building Foundation Mat (84.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

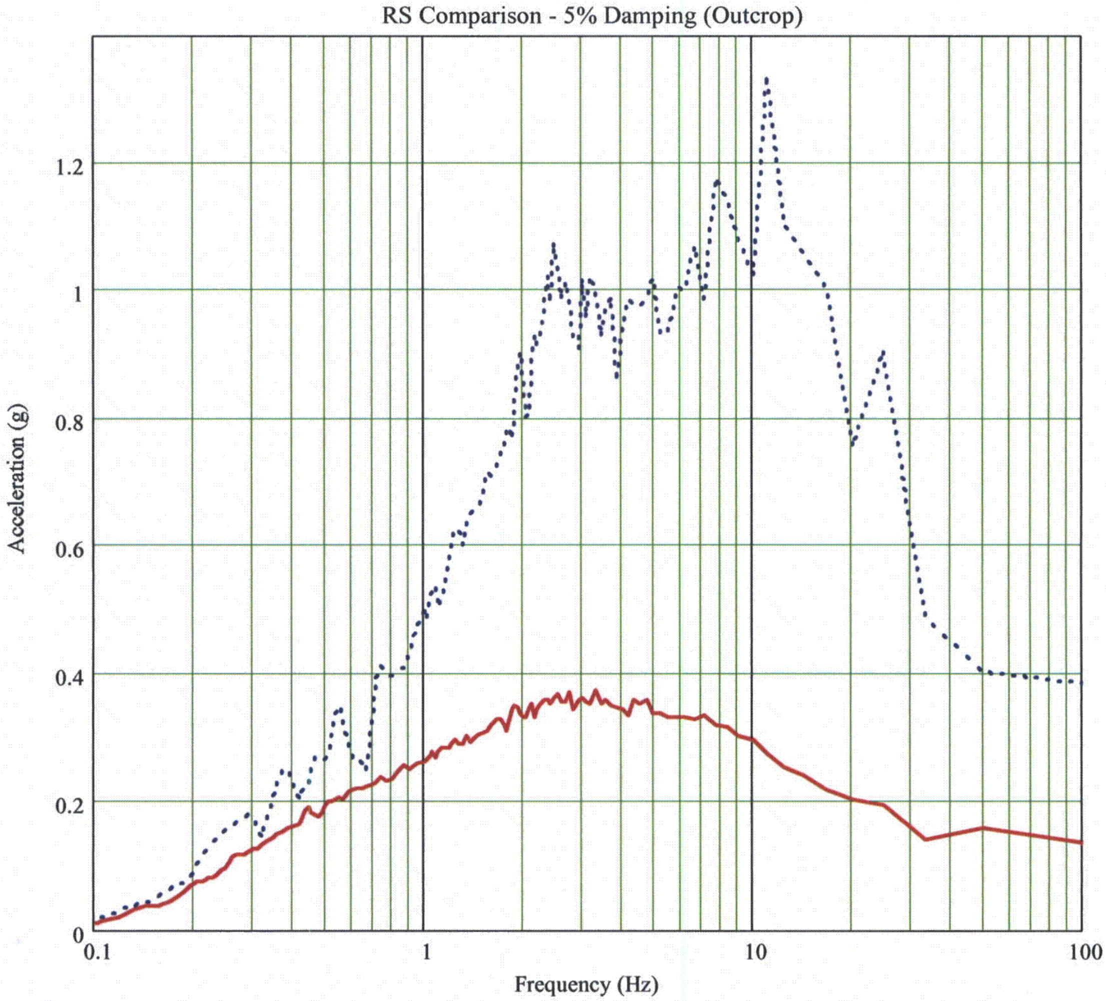
Figure 03.07.01-25l: In-Profile Spectral Comparison in EW Direction at Bottom of Reactor Building Mudmat (94.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

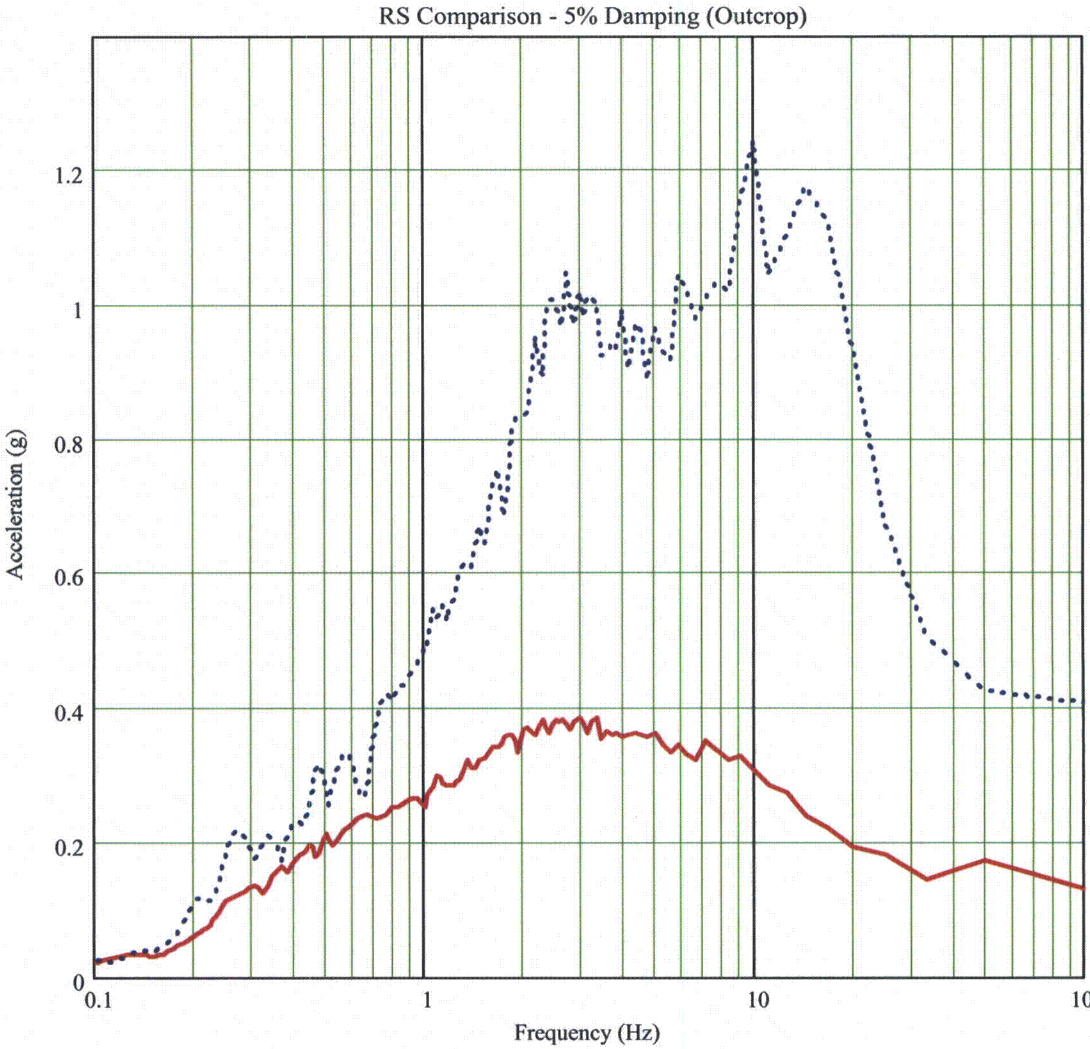
Figure 03.07.01-25m: Outcrop Spectral Comparison in NS Direction at Bottom of Reactor Building Foundation Mat (84.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

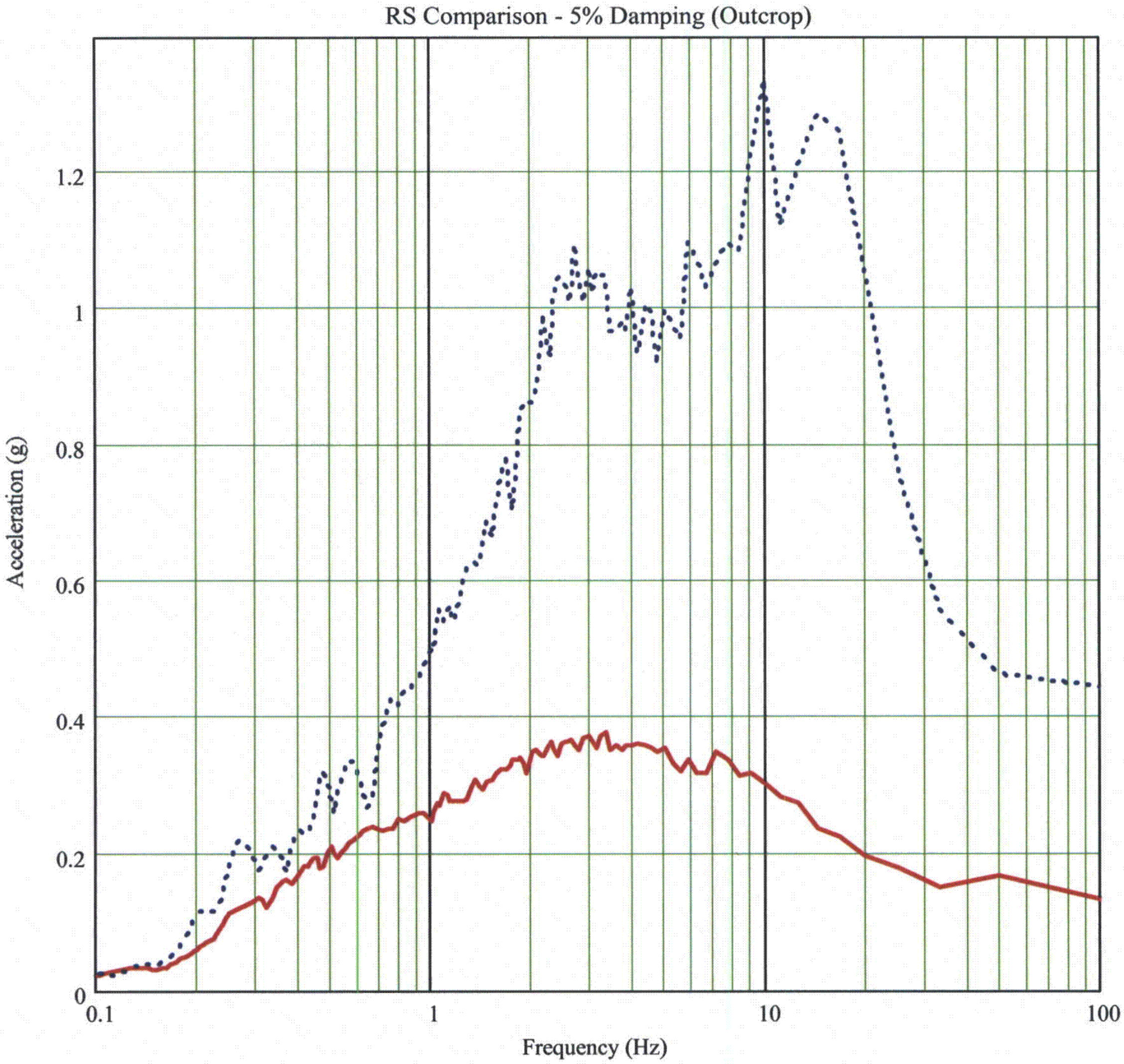
Figure 03.07.01-25n: Outcrop Spectral Comparison in NS Direction at Bottom of Reactor Building Mudmat (94.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

Figure 03.07.01-25o: Outcrop Spectral Comparison in EW Direction at Bottom of Reactor Building Foundation Mat (84.25 ft. below grade)



Note:

_____ Solid Red - Backfill Lower Bound Soil (0.13 g)
.... Dot Blue - DCD UB Soil Profile (0.30 g)

Figure 03.07.01-25p: Outcrop Spectral Comparison in EW Direction at Bottom of Reactor Building Mudmat (94.25 ft. below grade)

RAI 03.07.01-26

QUESTION:

Follow-up Question to RAI 03.07.01-18, Revision 1 (STP-NRC-100093)

10CFR50 Appendix S requires that seismic evaluation must take into account soil-structure interaction (SSI) effects. The applicant has provided the seismic soil pressure profiles between the RB and CB obtained from SSI analysis that include potential increase in the lateral pressures due to SSSI effects for the site-specific Safe Shutdown Earthquake (SSE). The calculated pressures are compared to those of DCD for the RB north wall in Figure RAI 03.07.01-18a and for the CB south wall in Figure RAI 03.07.01-18b. In evaluating the seismic soil pressures obtained from the SSI analysis, the staff does not find any details regarding the SSI model that incorporates the structure to structure interaction effect (SSSI). In order to complete this assessment, the applicant is requested to provide the SSI model and properties and describe in sufficient detail a) how the SSI analysis including the effects of SSSI was performed, b) how the input motions were defined, c) what software was used to perform this analysis, and d) how the results from input in three directions were combined. The applicant is also requested to include this description in the FSAR. The staff needs this information to determine that the effect of structure to structure interaction on seismic soil pressure at STP site is properly evaluated and bounded by the DCD design.

RESPONSE:

The seismic soil pressure profiles between the Reactor Building (RB) and Control Building (CB), obtained from Soil-Structure Interaction (SSI) analyses, including the effects of Structure-to-Structure Interaction (SSSI) are provided in RAI 03.07.01-18, Revision 1. The following provides additional details requested above:

a) To evaluate the effect of SSSI on the soil pressures for the RB and CB walls, two dimensional (2D) analyses of RB and CB individually, and 2D SSSI analyses of RB and CB together with Turbine Building (TB) were performed using SASSI2000 software. Since the RB and CB and non-category I TB are closely spaced in the North-South (N-S) direction, the SSSI analysis was performed in the N-S direction. Both the RB and CB were analyzed individually in the N-S direction. The SSI analysis was repeated for (1) the RB+CB model and (2) the RB+CB+TB model to consider the SSSI effects. The results of these SSI analyses were enveloped. The 2D models used for these analyses are similar to the models described in DCD Tier 2, Section 3A.9.7 for considering SSSI effect on the RB and CB and soil pressures on the building walls. For soil properties variation effects, each analysis was performed using three site-specific SSE strain-compatible in-situ soil conditions: upper bound, mean and lower bound, and the results were enveloped.

   The details of the N-S direction structural part of the SSI model of the RB + CB, and RB + CB + TB are shown in Figures 3A-299 and 3A-300, respectively (see the attached

COLA mark-up). In these figures, the elevation 39.37 feet corresponds to finished grade elevation of 12.00 meter TMSL noted in DCD, which corresponds to STP finished grade elevation of 34 feet MSL.

The RB is idealized by a center-line stick model of a series of massless beam elements representing the building walls, Reinforced Concrete Containment Vessel (RCCV), Reactor Shield Wall (RSW)/Pedestal and Reactor Pressure Vessel (RPV). Similar to the three dimensional model, the center-line stick model consists of three individual sticks, one for RB walls, one for RCCV and one for RSW/Pedestal with RPV supported on it. Axial, flexural, and shear deformation effects are included in beam elements properties. Coupling between individual structures is modeled by linear spring elements. Masses, including dead weights of the structural elements, equipment weights and piping weights, are lumped to nodal points. The weights of water in the spent fuel storage pool and the suppression pool are also considered and lumped to appropriate locations. The basemat and the mudmat are modeled by 4-node plain strain elements. To properly transfer the rotation of the stick model to the basemat (and vice-versa), a set of rigid beams are placed at the top of the basemat connecting each stick to its respective footprint. The stick representing the outer walls of the RB is connected to the side walls in horizontal directions by a set of rigid beams to reflect the direct connect condition of outside wall with the soil. The soil adjacent to the building is modeled by 4-node plane strain elements. The structural model properties (stiffness and mass) for the 2D model correspond to per unit depth (1 foot dimension in the out-of-plane direction) of the RB.

To assure that the 2D RB model reasonably represents the dynamic characteristics of the 3D RB model, the fixed base frequencies of the 2D RB model are compared with the fixed base frequencies of the 3D RB model provided in DCD Table 3.7-2 (N-S model). This comparison is provided in Table 03.07.01-26.1 and it shows that the frequencies compare reasonably well.

The CB is idealized by beam elements with lumped masses located at each floor elevation. The side walls are modeled with beam elements, which provide shear rigidity in the N-S direction. The basemat and the mudmat are modeled by 4-node plain strain elements. To properly transfer the rotation of the stick model to the base slab (and vice-versa), a set of rigid beams are placed at the bottom of the basemat connecting the stick to its footprint. The stick representing the walls is connected to cross walls in the horizontal direction by a set of axially rigid beams to reflect the direct contact condition of the outside wall with the soil. The soil adjacent to the building is modeled by 4-node plane strain elements. The structural model properties (stiffness and mass) for the 2D model correspond to per unit depth (1 foot dimension in the out-of-plane direction) of the CB.

Similar to the RB model, the fixed base frequencies of the 2D CB model are compared with the fixed base frequencies of the 3D CB model provided in DCD Table 3.7-5 (N-S model). This comparison is provided in Table 03.07.01-26.2 and it shows that the frequencies compare reasonably well.

The TB model consists of two concentric lumped-mass sticks representing the building structures and the turbine generator pedestal. The simple representation is sufficient since the TB representation is only to evaluate its effect on the CB and RB. Similar to the RB and CB 2D models, the structural model properties (stiffness and mass) of the TB correspond to per unit depth in the E-W direction.

Figure 03.07.01-26.1 and Figure 03.07.01-26.2 compare the basemat level response spectra calculated from the 2D RB+CB +TB SSI analysis and the 3D SSI analyses of the RB and CB, respectively. These comparisons show that the spectra compare well.

b) The site-specific input motion was defined at the grade elevation.

c) SASSI2000 software was used in the above analyses.

d) Since RB, CB and TB line-up along N-S direction, models are analyzed only in N-S direction for N-S direction input motion. This is similar to the SSSI analyses described in DCD Tier 2 Section 3A.9.7. SSSI effects in other two directions (E-W and Vertical) are expected to be insignificant.

During a telephone conference call between NRC staff and STPNOC, on August 31, 2010, NRC Staff mentioned that the seismic soil pressures provided in DCD Table 3A-18 and shown in Figures RAI 03.07.01-18a and RAI 03.07.01-18b in response to RAI 03.07.01-18, Revision 1, near the grade elevation, are substantially higher than the corresponding site-specific soil pressures. NRC Staff requested the reason for this substantially higher soil pressure in DCD Table 3A-18. The reason for the substantially higher soil pressure in DCD is that much stiffer soil profile was used in the DCD analyses, as compared to the STP 3&4 site-specific soil profile. The seismic soil pressures calculated in DCD Table 3A-18 are based on the enveloped pressures calculated for DCD soil profiles UB1D150, VP3D150, and VP5D150. The strain-compatible shear wave velocity for VP5D150 soil profile is about 2900 ft/sec as compared to the strain compatible shear wave velocity of 700 ft/sec for the STP 3&4 site-specific soil profile (upper bound soil profile near the ground surface). Thus, the soil profile for DCD case is much stiffer than the soil profile for the STP 3&4 site-specific case. As expected, in DCD case, a substantial part of the seismic shears from the above ground parts of the RB and CB structures are resisted by the stiffer soil layer near the grade elevation, thus producing higher seismic soil pressure.

Table 03.07.01-26.1
Fixed Base Reactor Building Model
Frequencies

Mode No.	Frequency (HZ)	
	DCD Table 3.7-2	2D Model
1	4.14	4.14
2	4.53	4.64
3	7.71	8.29
4	9.01	10.22
5	9.6	11.91
6	10.1	12.11
7	11.53	13.21
8	12.72	13.35
9	13.44	17.08
10	13.58	17.18
11	14.64	17.71
12	15.6	18.67
13	17.46	18.85
14	18	18.88
15	18.95	20.81
16	22.01	21.57
17	22.72	22.12
18	24.31	24.26
19	25.48	26.52
20	26.11	27.29
21	27.08	27.71
22	28.2	29.23
23	29.84	32.75
24	30.94	---
25	33.16	---

Table 03.07.01-26.2
Fixed Base Control Building Model
Frequencies

Mode No.	Frequency (HZ)	
	DCD Table 3.7-5	2D Model
1	5.59	6.29
2	15.91	17.71
3	29.22	29.63
4	30.85	---

Figure 03.07.01-26.1: Reactor Building Base Slab 5% Damped North-South In-Structure Response Spectra

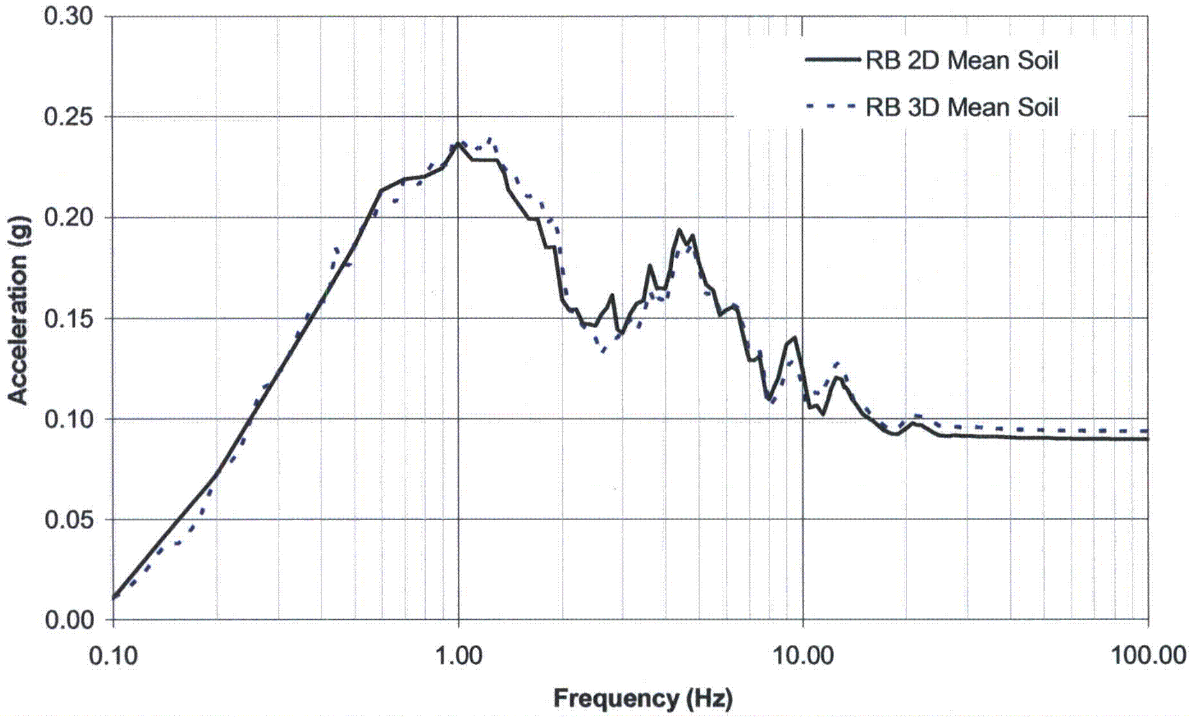
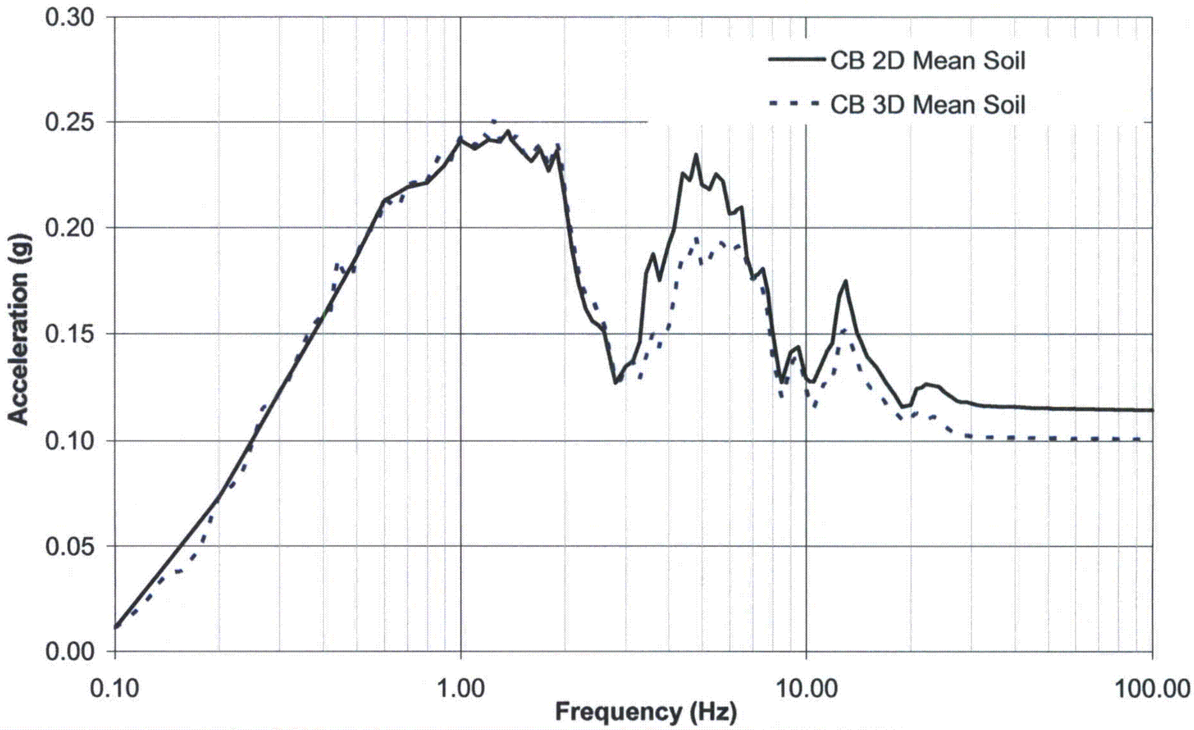


Figure 03.07.01-26.2: Control Building Base Slab 5% Damped North-South In-Structure Response Spectra



COLA Part 2, Tier 2, will be revised to add a new Section 3A.21 and new Figures 3A-299 thru 3A-302 and renumber subsequent subsections as shown below.

3A.21 Soil Pressure on Reactor and Control Building Walls Considering Structure-to-Structure Interaction (SSSI) Effect

To evaluate the effect of SSSI on soil pressures on the RB and CB walls, two dimensional (2D) analyses of RB and CB individually, and 2D SSSI analyses of RB and CB together with Turbine Building (TB) were performed using SASSI2000 software. Since the RB and CB and non-category I TB are closely spaced in the North-South (N-S) direction, the SSSI analysis was performed in the N-S direction. Both the RB and CB were analyzed individually in the N-S direction. The Soil-Structure Interaction (SSI) analysis was repeated for (1) the RB+CB model and (2) the RB+CB+TB model to consider the SSSI effects. The results of these analyses were enveloped. The 2D models used for these analyses are similar to the models described in DCD Tier 2, Section 3A.9.7 for considering SSSI effect on the RB and CB and soil pressures on the building walls. For soil properties variation effects, each analysis was performed using three site-specific SSE strain-compatible in-situ soil conditions: upper bound, mean and lower bound, and the results were enveloped. The site-specific SSE input motion is defined at the grade elevation.

The details of the N-S direction structural part of the SSI model of the RB + CB, and RB + CB + TB are shown in Figures 3A-299 and 3A-300, respectively. In these figures, the elevation 39.37 feet corresponds to finished grade elevation of 12.00 meter TMSL noted in DCD, which corresponds to STP finished grade elevation of 34 feet MSL.

The RB is idealized by a center-line stick model of a series of massless beam elements representing the building walls, Reinforced Concrete Containment Vessel (RCCV), Reactor Shield Wall (RSW)/Pedestal and Reactor Pressure Vessel (RPV). Similar to the three dimensional model, the center-line stick model consists of three individual sticks, one for RB walls, one for RCCV and one for RSW/Pedestal with RPV supported on it. Axial, flexural, and shear deformation effects are included in beam elements properties. Coupling between individual structures is modeled by linear spring elements. Masses, including dead weights of the structural elements, equipment weights and piping weights, are lumped to nodal points. The weights of water in the spent fuel storage pool and the suppression pool are also considered and lumped to appropriate locations. The basemat and the mudmat are modeled by 4-node plain strain elements. To properly transfer the rotation of the stick model to the basemat (and vice-versa), a set of rigid beams are placed at the top of the basemat connecting each stick to its respective footprint. The stick representing the outer walls of the RB is connected to the side walls in horizontal directions by a set of rigid beams to reflect the direct connect condition of outside wall with the soil. The soil adjacent to the building is modeled by 4-node plane strain elements. The structural model properties (stiffness and mass) for the 2D model correspond to per unit depth (1 foot dimension in the out-of-plane direction) of the RB.

The CB is idealized by beam elements with lumped masses located at each floor elevation. The side walls are modeled with beam elements, which provide shear rigidity in the N-S direction. The basemat and the mudmat are modeled by 4-node plain strain elements. To properly transfer the rotation of the stick model to the base slab (and vice-versa), a set of rigid beams are placed at the bottom of the basemat connecting the stick to its footprint. The stick representing the walls is connected to cross walls in the horizontal direction by a set of axially rigid beams to reflect the direct contact condition of the outside wall with the soil. The soil adjacent to the building is modeled by 4-node plane strain elements. The structural model properties (stiffness and mass) for the 2D model correspond to per unit depth (1 foot dimension in the out-of-plane direction) of the CB.

The TB model consists of two concentric lumped-mass sticks representing the building structures and the turbine generator pedestal. The simple representation is sufficient since the TB representation is only to evaluate its effect on the CB and RB. Similar to the RB and CB 2D models, the structural model properties (stiffness and mass) of the TB correspond to per unit depth in the E-W direction.

Figures 3A-301 and 3A-302 provide the soil pressure profiles between the RB and CB obtained from SSSI analysis for site-specific Safe Shutdown Earthquake (SSE) along with the design soil pressures reported in DCD Table 3A-18 and Figures 3H.1-11 and 3H.2-14. As can be seen from these figures, the soil pressure profiles from the SSSI analysis are bounded by the envelope of the certified design soil pressures from DCD Table 3A-18 and Figures 3H.1-11 and 3H.2-14 with one exception. The soil pressure from the SSSI analysis for the CB slightly exceeds the certified design soil pressure at a depth of about 26 to 30 feet below the ground surface. At all other elevations the DCD soil pressures are higher than the site-specific soil pressure. Therefore, the total force due to the certified design soil pressure on the wall panel above or below it will be significantly higher than the total force due to soil pressure from the SSSI analysis. Therefore, the design based on certified design soil pressures is adequate.

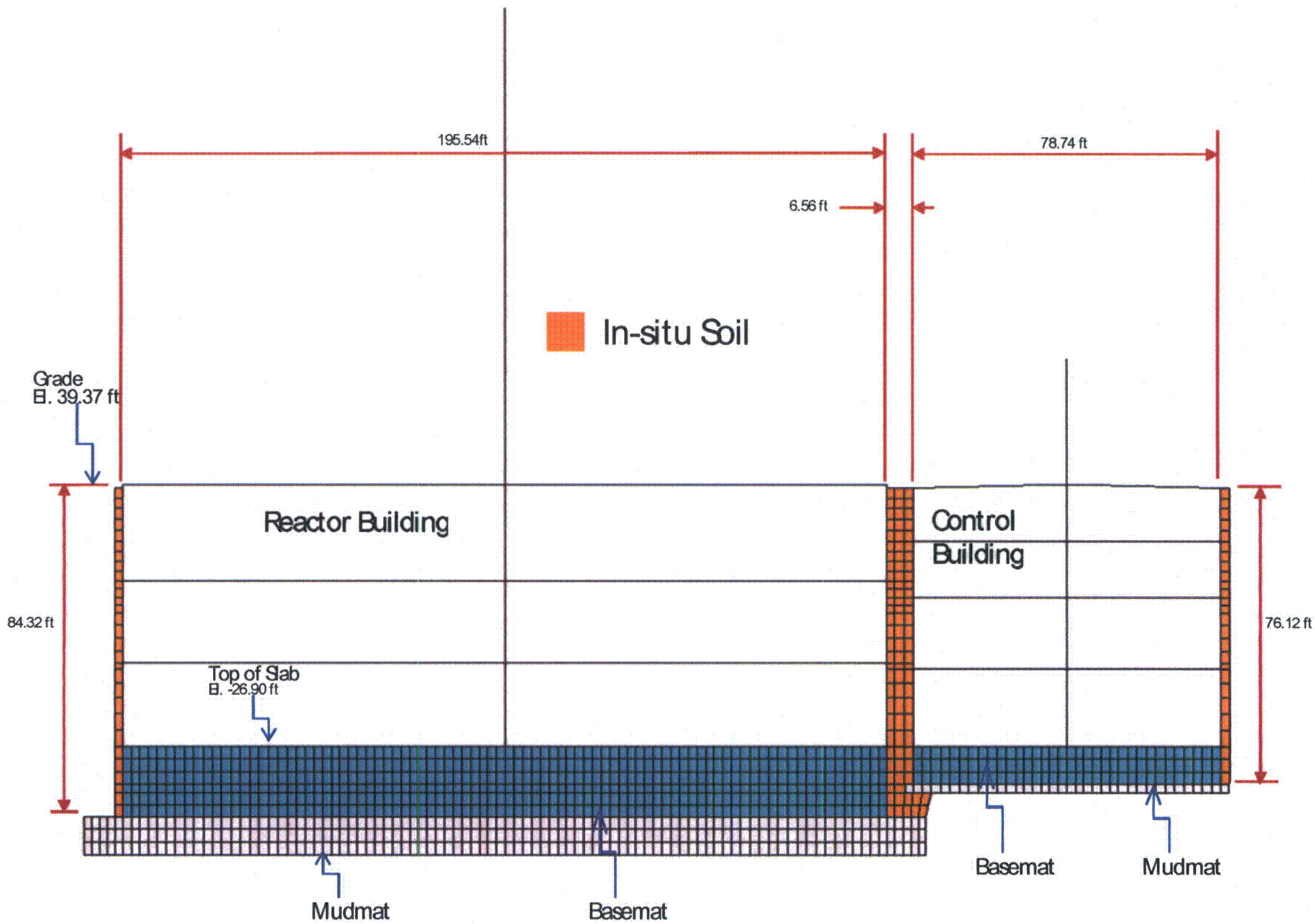


Figure 3A-299: RB+CB Model

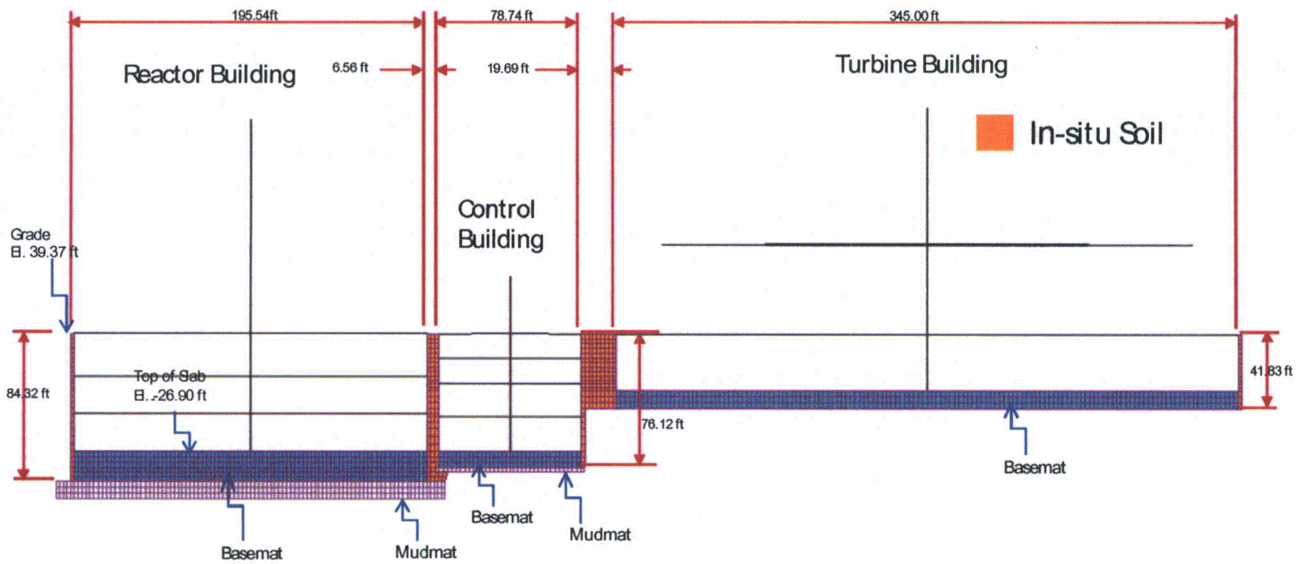


Figure 3A-300: RB+CB+TB Model

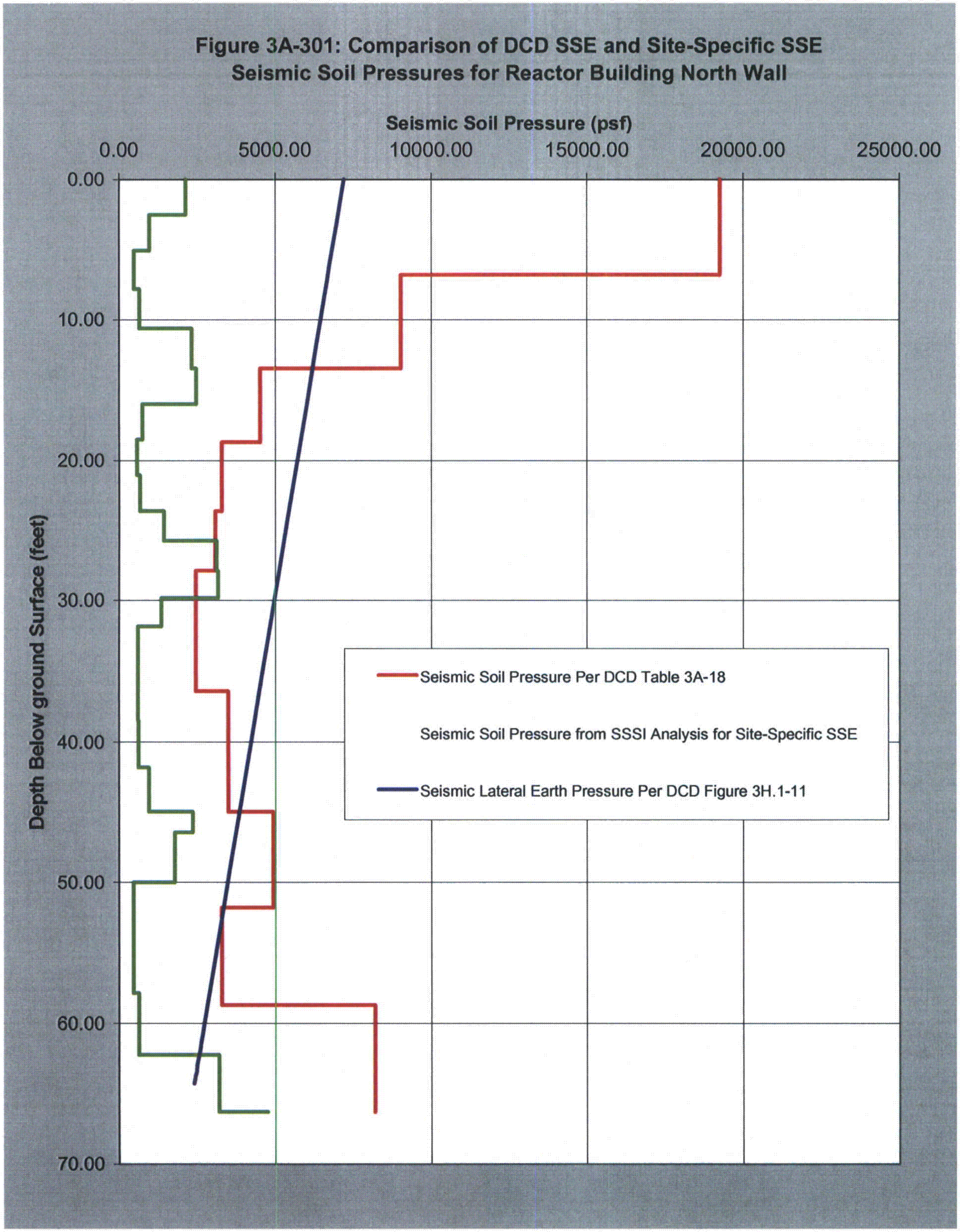


Figure 3A-301: Comparison of DCD SSE and Site-Specific SSE
Seismic Soil Pressures for Reactor Building North Wall
Seismic Soil Pressure (psf)
0.00
5000.00
10000.00
15000.00
20000.00
25000.00
Depth Below ground Surface (feet)
0.00
10.00
20.00
30.00
40.00
50.00
60.00
70.00
Seismic Soil Pressure Per DCD Table 3A-18
Seismic Soil Pressure from SSSI Analysis for Site-Specific SSE
Seismic Lateral Earth Pressure Per DCD Figure 3H.1-11

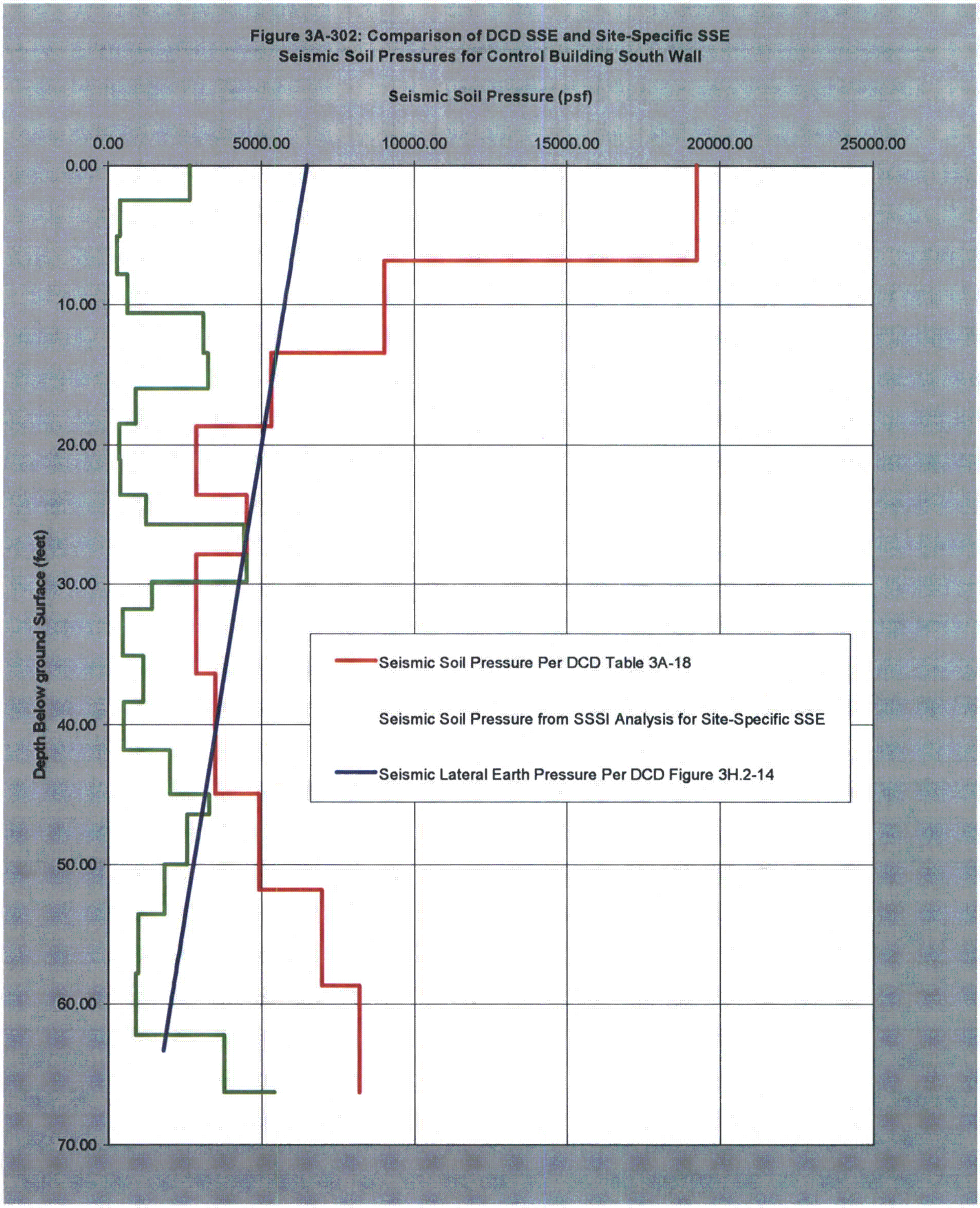


Figure 3A-302: Comparison of DCD SSE and Site-Specific SSE
Seismic Soil Pressures for Control Building South Wall
Seismic Soil Pressure (psf)
0.00
5000.00
10000.00
15000.00
20000.00
25000.00
0.00
10.00
20.00
30.00
40.00
50.00
60.00
70.00
Depth Below ground Surface (feet)
Seismic Soil Pressure Per DCD Table 3A-18
Seismic Soil Pressure from SSSI Analysis for Site-Specific SSE
Seismic Lateral Earth Pressure Per DCD Figure 3H.2-14

RAI 03.07.01-27

QUESTION:

Follow-up Question to RAI 03.07.01-19 (STP-NRC-100093)

1. 10CFR50, Appendix S requires that evaluation for SSE must take into account soil-structure interaction (SSI) effects and the expected duration of vibratory motion. In the response to the first paragraph of RAI 03.07.01-19, the applicant has presented its approach for developing the input motion for the SSI analysis and design of the DGFOSV that takes into account the impact of the nearby heavy RB and RSW Pump House structures. The applicant also stated that *"Conservatively, a 3-dimensional SAP2000 response spectrum analysis was used to obtain the safe-shutdown earthquake (SSE) design forces due to structure inertia. The seismic induced dynamic soil pressure on DGFOSV walls were computed using the method of ASCE 4-98, Subsection 3.5.3.2"* The response, however, does not provide details as to how the SSI analysis of the DGFOSV are performed and how the input motion developed are subsequently specified in the SSI analysis of DGFOSV to develop the structural response and in-structure response spectra for any equipment and subsystems within DGFOSV. From the response it appears that the applicant has not included explicitly DGFOSV structural model in the SASSI model of the RB and RSW Pump House structures to properly evaluate the SSSI effect on the DGFOSV. In order for the staff to determine if the evaluation of DGFOSV for SSE has appropriately accounted SSI effects, the applicant is requested to provide in the FSAR the following information:

   (a) Describe in detail the method used for the SSI analysis of DGFOSV including the procedures for treatment of strain dependent backfill material properties in the model, input motion used and how it is specified in the analysis, variation of soil properties, and the computer programs used for SSI analysis.

   (b) Describe in detail how SAP2000 analysis of DGFOSV was performed including, how foundation soil/backfill material was represented, how many modes were extracted, what modal damping values were used, how the input motion was specified, and what type of boundary conditions were used.

   (c) Demonstrate that the DGFOSV foundation response spectra and dynamic soil pressure (on DGFOSV basement walls using ASCE 4-98 criteria) used in the design of DGFOSV will envelop the results of structure to structure (SSSI) interaction analysis which explicitly models DGFOSV structure in the SSI model of RB and the RSW Pump House structure.

   (d) Describe in detail if there is any Category I tunnel structure for transporting Diesel Fuel Oil between DGFOSV and the Diesel Generator located in other buildings including its layout and configuration and seismic analysis and design method.

2. In the response to Item 2 of RAI 03.07.01-19, the applicant has stated that the P-wave damping ratios are assigned the same values as those calculated for the S-wave damping ratios because of the **upcoming** recommendations of ASCE 4-09 standards. It is further stated that this recommendation is based on the recent observation of earthquake data and the realization that the waves generated due to SSI effects are mainly surface and shear waves. It is noted that the NRC has not endorsed ASCE 4-09 for estimating the P-wave damping. In general, the P-wave damping is primarily associated with the site response rather than SSI effects. Because the Pwave energy for the most part will travel in water within the saturated soil media at relatively high propagation speed and is not affected by shear strains of degraded soil, the P-wave damping will be small. As such, the applicant is requested to provide quantitative assessment by performing sensitivity analysis that shows that seismic responses of Category I structures are not adversely affected to a lower P-wave damping.

RESPONSE:

The following provides the response to part 2 of this RAI. The response to parts 1a through 1c will be provided in a supplemental response by November 1, 2010 and the response to part 1d will be provided in another supplemental response by November 15, 2010.

2. The adequacy of assigning P-wave damping ratios the same values as those calculated for the S-wave damping ratios is examined in Reference 1 (below). In this study, the ground motion recordings at two downhole arrays were utilized (Lotung array in Taiwan and the Port Island array in Japan). The study examined two different cases. In the first case, the soil damping used for vertical wave propagation, associated with P-waves, is the same as calculated from site response analysis for horizontal excitation. In the second case smaller damping is used for vertical wave propagation. The results were compared in each case to the recorded vertical motions. The study concluded that the use of smaller damping results in over-estimating the response spectra, and that the use of the S-wave damping ratios for vertical wave propagation results in good agreement with the recorded motions. The study recommends the use of S-wave damping, resulting from site response analysis for horizontal excitation, in vertical wave propagation analysis, with an upper limit of 10%.

   For the STP site, the S-wave damping ratios calculated in the site response analysis are relatively small (in the range of 1.5% and not exceeding 3%); and, in light of the referenced study, use of the S-wave damping is confirmed to be an adequate representation of P-wave damping.

   Furthermore, following the industry practice, the vertical motion at the site is calculated using the vertical-to-horizontal (V/H) acceleration response spectra ratio at the foundation elevation, and not through site response analysis of vertical excitation. Also, the deconvolved vertical motion at the Reactor Building and Control Building foundation outcrop, in free field, with the site specific Safe Shutdown Earthquake specified at ground surface envelops the Foundation Input Response Spectra by a wide margin (See COLA Figures 3A-235 and 3A-244).

Therefore, the assigned P-wave damping, which is primarily associated with site response rather than Soil Structure Interaction effects as stated in the RAI, does not affect the vertical motion at the STP site. The use of the shear wave damping for P-wave damping is consistent with the approach in the ABWR DCD, Tier 2, Section 3A.6.

No additional COLA revision is required as a result of this response.

References:

1. Mok, C. M., Chang, C.-Y., and Legaspi, D. E. (1998) "Site Response Analyses of Vertical Excitation," Geotechnical Earthquake Engineering and Soil Dynamics III, Geotechnical Special Publication No. 75, Proceedings of a Specialty Conference, Vol.1, pp.739-753, University of Washington, Seattle Washington, August 3-6, 1998. (Attached)

U7-C-STP-NRC-100208
Attachment 7

RAI 03.07.01-27

GEOTECHNICAL SPECIAL PUBLICATION NO. 75

GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS III

Volume 1

PROCEEDINGS OF A SPECIALTY CONFERENCE

SPONSORED BY
Geo-Institute of the American Society of Civil Engineers

CO-SPONSORED BY
US Air Force Office of Scientific Research

August 3–6, 1998
University of Washington
Seattle, Washington

EDITED BY
Panos Dakoulas
Mishac Yegian
Bob Holtz

GEO INSTITUTE

ASCE American Society of Civil Engineers
1801 ALEXANDER BELL DRIVE
RESTON, VIRGINIA 20191–4400

reduced for liquefiable deposits of limited areal extent. Ground oscillation, including strong accelerations and large relative displacements, is likely to occur by compressive wave transmission through the surface layer if the earthquake shaking exceeds thresholds presented herein. The paper does not address damage that could potentially occur independent of dynamic motions (i.e. sand boils, post-liquefaction consolidation settlement, or bearing failures of footings) or that could occur due to pore pressure migration. A future research goal is to determine whether the limited area of a liquefiable deposit can sufficiently limit the strain energy such as to prevent liquefaction.

REFERENCES

Bartlett, S. F., and L. Youd (1992). "Empirical Prediction of Lateral Spread Displacement." *Fourth Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures for Soil Liquefaction*, Honolulu, Hawaii, 351 - 365.

Byrne, P.M. and M. Beatty (1997). "Liquefaction Induced Displacements." *Fourteenth International Conference on Soil Mechanics and Foundation Engineering*, Hamburg, Germany, 185 – 194.

Housner, G. W., editor, (1985). *Liquefaction of Soils During Earthquakes*, Committee on Earthquake Engineering, National Academy Press, Washington, DC.

Hamada. M. and S. Shimizu (1992). "Large Ground Deformations and Their Effects on Lifelines: 1983 Nihonkai-Chubu Earthquake" in *Case Studies of Liquefaction and Lifeline Performance During Past Earthquakes, Volume 1*, Technical Rep. NCEER-92-001, National Center for Earthquake Engineering Research, Buffalo, New York.

Itasca Consulting Group (1993). "FLAC: Fast Lagrangian Analysis of Continua," Program Software Version 3.22, Minneapolis, Minnesota.

O'Rourke, T. D., and Pease, J. (1997). "Mapping Liquefiable Thickness for Seismic Hazard Assessment." *J. of Geotech. Engnrg.*, 123(1), pp. 46 - 56.

Pease, J. W., and O'Rourke, T. (1995). "Liquefaction Hazards in the San Francisco Bay Region: Site Investigation, Modeling, and Hazard Assessment at Areas Most Seriously Affected by the 1989 Loma Prieta Earthquake." *Final Report, U.S.G.S. Grant 1434-93-G-2332*, Cornell University, Ithaca, New York.

Pease, J. W., and O'Rourke, T. (1997). "Seismic Response of Liquefaction Sites." *J. Geotech. Engnrg.*, 123(1), pp. 37 - 45.

Youd, T. L., and Holzer, T. (1994). "Piezometer performance at Wildlife liquefaction site, California." *J. Geotech. Engnrg.*, 120(6), pp. 975 - 995.

Youd, T. L., and Keefer, D. (1994). "Liquefaction during the 1977 San Juan Province, Argentina earthquake (Ms = 7.4)." *Engineering Geology*, 37, 211-233.

Zeghal, M., and Elgamal, A. (1994). "Analysis of site liquefaction using earthquake records." *J. Geotech. Engnrg.*, 120(6), pp. 996 - 1017.

SITE RESPONSE ANALYSES OF VERTICAL EXCITATION

Chin Man Mok[1], M. ASCE, C.-Y. Chang[2], M. ASCE and Dante E. Legaspi, Jr.[3], M. ASCE

ABSTRACT

In this study, the ground motion recordings at two downhole arrays (Lotung downhole array in Taiwan and the Port Island downhole array in Japan) were utilized (1) to back-calculate the compression-wave velocities (Vp) during strong seismic events and (2) to examine the effects of soil damping on vertical site response. The back-calculation of Vp was based on soil column fundamental frequencies identified from the Fourier spectral ratios. The back-calculated Vp of the near-surface unsaturated soils (even below the groundwater table) are as much as 40 to 60 percent less than values determined from geophysical measurements. For soil layers that have high Vp (close to or higher than that of water), the back-calculated Vp are in good agreement with the geophysical measurements. Parametric vertical site response analyses were performed for a range of damping ratio values using the back-calculated Vp and the ground motion recordings at depth as control motions. The response spectra of the computed and the recorded motions are in good agreement when the compression-wave damping ratios used are equal to the geometric mean of the strain-compatible shear-wave damping ratios (estimated from horizontal site response analyses), but limited to not more than 10 percent. These dynamic soil behaviors are believed to be related to the degree of saturation.

INTRODUCTION

Site response effects on vertical ground motions are generally incorporated using the same analytical procedure as used for horizontal ground motions. For horizontal excitation, the dynamic soil properties affecting the soil response include shear-wave velocity (or shear modulus), soil damping, and mass density. For vertical excitation,

[1] Senior Engineer, Geomatrix Consultants, Inc., 100 Pine Street, 10th Floor, San Francisco, CA 94111
[2] Principal Engineer, Geomatrix Consultants, Inc., 100 Pine Street, 10th Floor, San Francisco, CA 94111
[3] Project Engineer, Geomatrix Consultants, Inc., 100 Pine Street, 10th Floor, San Francisco, CA 94111

the soil parameters affecting the soil response are same as those for the horizontal excitation except for wave velocity. For vertical excitation, the compressional-wave velocity rather, than the shear-wave velocity controls the soil response. For horizontal excitation, it has been well established that shear-wave velocity or shear modulus decreases and soil damping increases with induced shear strain due to nonlinear soil response (Seed and Idriss, 1970). For vertical excitation, little research has been conducted into how the compressional-wave velocity or the constrained modulus and soil damping vary with levels of shaking. The objective of this study was to develop procedures for conducting site response analysis of vertical excitation based on evaluating site response at two downhole array sites (the Lotung downhole array in Taiwan and the Port Island downhole array in Japan) for which strong motion recordings at the ground surface and at depths are available. The scope of the study includes backcalculating compressional-wave velocity from the recordings of vertical motion and performing parametric studies to examine effects of variations in compressional-wave velocity and soil damping on the vertical motions. This paper presents the results of the study and recommendations for site response analysis procedures for vertical excitation.

SITE RESPONSE ANALYSIS USING DOWNHOLE ARRAY RECORDINGS FROM LOTUNG, TAIWAN

Chang et al. (1994) backcalculated compressional-wave velocities of soil layers and evaluated effects of soil damping on soil response due to vertical excitation using recordings from the downhole array at Lotung, Taiwan. The procedure for using the downhole array recordings to backcalculate shear- or compressional-wave velocities of soil layers by identifying the fundamental frequency of the soil column in conjunction with wave propagation theory is described in Chang et al. (1991, 1996). The study concluded that the backcalculated compressional-wave velocities for near-surface soil layers (i.e., unsaturated soils in the upper 10 m below the ground surface) may be less than those measured using geophysical techniques, and high soil damping equal to average values from the two horizontal-component excitation (ranging from 5 to 10 percent and higher than a nominal value of 2 percent) may be more appropriate for calculating site response due to vertical excitation. Additional analyses to examine effects of parametric variations in compressional-wave velocity and soil damping on the vertical motion were performed as part of this study.

Lotung Downhole Array Site

An upper layer about 30 to 55 m thick at the Lotung array site consists predominantly of silty sand and sandy silt containing some gravel. The soil beneath this layer consists predominantly of clayey silt and silty clay to a depth of about 400 m. The water table is within 1 m of the ground surface.

A geophysical measurement program was conducted to assess site shear-wave and compressional-wave velocities (Anderson, 1993). Measured shear-wave velocity, shown on Fig. 1, increases gradually from approximately 110 m/s at the ground surface to approximately 200 to 220 m/s at a depth of approximately 18 m. Below this depth, the shear-wave velocity increases gradually to approximately 250 to 280/ms at a depth of 60 m. Below 60 m, the results of uphole testing indicate shear-wave velocities of 320 m/s at depths of 60 to 80 m and 480 m/s at depths of 80 to 150 m.

Measured compressional-wave velocities, also shown on Fig. 1, in the upper 10 m layers increase from about 350 m/s near the ground surface to about 1400 m/s at a depth of 10 m. Below 10 m, compressional-wave velocities are relatively constant, ranging from 1250 to 1500 m/s. Note that, even though the water table was near the ground surface and site soils were below the water table, the compressional-wave velocities of the soils above a depth of 10 m were lower than that of water, which is about 1500 m/s. Allen et al. (1980) indicated that the compressional-wave velocities of soils are strongly affected by the compressibility of the soil and fluid components of the soil-fluid system. The compressional-wave velocity decreases dramatically even slightly below full saturation. Thus, it may be inferred that the soils in the upper 10 m at Lotung were not fully saturated.

Site Response Analysis for Vertical Excitation

The Lotung downhole array consists of a surface accelerometer and accelerometers at depths of 6, 11, 17, and 47 m. Chang et al. (1994) backcalculated compressional-wave velocities for soil layers between depths of 0 to 6 m, 6 to 11 m, 11 to 17 m, and 17 to 47 m from Fourier spectral ratio analyses of the downhole recordings from events LSST12 and lSST16 summarized in Table 1 (Fig. 2). The backcalculated compressional-wave velocities in the upper 17 m were substantially lower than the values determined from the geophysical measurements (most likely because of unsaturated soils). For the soil layer between 6 to 17 m, the compressional-wave velocity was less by about 40 to 50 percent. Below a depth of 17 m, the backcalculated compressional-wave velocities are slightly higher than the geophysical measurements.

Parametric studies of site response for vertical excitation were performed (also by Chang et al. [1994]) to evaluate appropriate soil damping for use in site response analyses. In these analyses, the recorded motions at a depth of 17 m were used as input motions. Motions were computed for the ground surface and other depths, then compared with the recorded motions.

TABLE 1
Lotung Earthquake Ground Motion Data Analyzed

Earthquake	Date	Magnitude	Distance (km)	Depth (km)	Azimuth (deg)	Peak Ground Acceleration, (g's)		
						EW	NS	Vert
LSST12	7/30/86	6.2	5.2	1.6	131	0.16	0.19	0.20
LSST16	11/14/86	7.0	77.9	6.9	174	0.13	0.17	0.10

* Recorded at ground surface station FA1-5.

For events LSST12 and LSST16, parametric studies were conducted for the four cases in Table 2.

TABLE 2
Site Response Analyses at Lotung Site

Case	Compressional-wave velocity used	Soil damping used
1	Inferred from Fourier spectral ratio analyses	Soil damping equals to average values from site response analyses for horizontal excitation
2	Inferred from Fourier spectral ratio analyses	2 percent
3	Geophysical measurements	Soil damping equals to average values from site response analyses for horizontal excitation
4	Geophysical measurements	2 percent

The soil damping ratios inferred from the site response analyses for the horizontal motion range from 5 to 8 percent for event LSST12 and from 5 to 10 percent for event LSST16 (Chang et al., 1991). Comparisons of the response spectra (5-percent damped) of the computed and recorded vertical motions for the four cases analyzed are shown on Figs. 3 to 6 for event LSST16. Similar results were obtained for event LSST12. The results for events LSST12 and LSST16 indicate that use of compressional-wave velocity inferred from the recordings and average values of soil damping estimated from the horizontal excitation (Case 1) generally results in better agreement between the computed and recorded motions (Fig. 3). When the compressional-wave velocities inferred from geophysical measurements were used in conjunction with average values of soil damping estimated from the horizontal excitation (Case 3, Fig. 5), the comparisons were not as good as those for Case 1, especially near the ground surface - there was a reduction in compressional-wave velocity of the soils near the ground surface that was not incorporated in the analyses. When a 2-percent soil damping ratio was used for the entire soil profile, the response was significantly overestimated, especially at the fundamental frequency of the soil column (Fig. 4 and 6 for event LSST16).

SITE RESPONSE ANALYSIS USING THE DOWNHOLE ARRAY RECORDINGS FROM PORT ISLAND, JAPAN

A downhole array is located on Port Island, a reclaimed island off the city of Kobe, Japan. The array recorded ground motions at the ground surface and at depths of 16, 32, and 83 m during the Hyogo-Ken Nanbu (Kobe) earthquake (M_w 6.9) of January 17, 1995. Vertical components of the downhole recordings were analyzed to evaluate appropriate compressional-wave velocities and soil damping for use in site response analyses of vertical excitation.

Site response analyses of vertical motion using a soil damping ratio of 10 percent or higher produced better agreement between recorded and computed motions than using damping ratios lower than 10 percent, a conclusion similar to that derived from the study of the Lotung downhole array data described above.

Port Island Downhole Array Site

The Port Island downhole array site is underlain by about 19 m of loose fill (decomposed, weathered granite fill) overlying alluvial and diluvial deposits. The natural deposits are comprised of : approximately 8 m of soft clay overlying about 6 m of sands and gravels that are underlain by interbedded layers of sands, gravels, and clays. The depth of the water table at the time of the Kobe earthquake was estimated to be about 4 m below ground surface. The fill below the water table and above the soft clay layer (at a depth of about 19 m) are believed to have liquefied during the Kobe earthquake, as evidenced by widespread sand boils on the island. Shear-wave and compressional-wave velocity profiles along the soil profile from CEORKA (1995) and Iwasaki (1995) are shown on Fig. 7. Note that compressional-wave velocities at depths above approximately 32 m are lower than that of water, (except at depths between 12 to 19 m, where the compressional-wave velocity is approximately equal to that of water), even though the groundwater table was estimated to be at a depth of 4 m). The soils at depths above 32 m probably were not fully saturated, resulting in compressional-wave velocities lower than that of water.

Site Response Analyses for Vertical Excitation

The instruments installed at the site consist of synchronized accelerometers oriented in the N00E, N90E, and vertical directions. They are located on the ground surface and at depths of 16, 32, and 83 m in the free field. As with the analyses conducted for the Lotung downhole array data described in Chang et. al. (1994), Fourier ratio analyses were performed to backcalculate compressional-wave velocity from the recordings. Using the procedure described previously for analyzing the Lotung downhole array data, backcalculated compressional-wave velocities were compared with those from the geophysical measurements (Fig. 13). The wave velocities in the upper 16 m are lower than those from the geophysical measurements (an average reduction of about 50 % in the upper 16 m). Backcalculated wave velocities at depths deeper than 16 m are slightly higher than those from geophysical measurements.

Elgamal et al. (1996) analyzed the horizontal ground motions recorded at the Port Island downhole array site during the Hyogokey-Nanbu earthquake of January 17, 1995. Shear-strain time histories between two recording stations were backcalculated from the displacement time histories. Based on the backcalculated peak shear strain for each soil layer, values of soil damping ratio corresponding to the horizontal excitation were estimated from the Seed and Idriss (1970) lower-bound damping curve for sands and the Vucetic and Dobry (1991) damping curve for clays with PI=30. The estimated values of soil damping ratio are 16 percent for the soil layer between 0 and 16 m, 10 percent between 16 and 32 m, and 8 percent between 32 and 83 m depths.

Parametric site response analyses using the motion recorded at a depth of 83 m as an input motion were conducted for the five cases in Table 3.

TABLE 3
Site Response Analyses at Port Island Site

Case	Compressional-wave velocity used	Soil damping used
1	Inferred from Fourier spectral ratio analyses	Soil damping equals to average values from site response analyses for horizontal excitation
2	Inferred from Fourier spectral ratio analyses	From site response analyses of horizontal excitation but limited to 10 percent
3	Inferred from Fourier spectral ratio analyses	5 percent
4	Geophysical measurements	From site response analyses of horizontal excitation but limited to 10 percent
5	Geophysical measurements	5 percent

Comparisons of the response spectra (5-percent damped) of the computed and recorded vertical motions for the five cases are shown on Figs. 8 to 12. The results shown on Figs. 8 and 9 indicate that use of compressional-wave velocity inferred from the recordings and soil damping estimated from the horizontal excitation (Case 1) or soil damping values limited to 10 percent (Case 2) generally results in good agreement between computed and recorded motions. When the compressional-wave velocities inferred from the geophysical measurements were used in conjunction with soil damping estimated from the horizontal excitation but limited to 10 percent (Case 4, Fig. 11), the response spectra of the computed motions are generally equal to or higher than those of the recorded motions except at some isolated periods at the ground surface. The differences between the response spectra of the computed and

recorded motions at the ground surface are due primarily to a reduction in compressional-wave velocity near the ground surface that was not accounted for in the model. For the cases that used an overall soil damping ratio of 5 percent in conjunction with compressional-wave velocities inferred from either the Fourier spectral ratio analysis or the geophysical measurements (Cases 3 and 5), the response spectra of the computed motions are generally higher than those of the recorded motions (Figs. 10 and 12).

DEVELOPMENT OF RECOMMENDED PROCEDURE

The analyses of the vertical motions recorded at the Lotung and Port Island downhole array sites conducted as part of this study indicate that the compressional-wave velocity of the near-surface unsaturated soils (even below the groundwater table) may be as much as 40 to 60 percent less than the values determined by geophysical measurements. For those soil layers that exhibit high compressional-wave velocities (close to or higher than that of water), no reduction in compressional-wave velocity due to earthquake excitation was observed. The backcalculated compressional-wave velocities are compared with values from the geophysical measurements for both the Lotung and Port Island array sites on Fig. 13.

Figure 13 also shows a recommended relationship between the backcalculated values and the geophysical values. The recommended relationship indicates that compressional-wave velocities are reduced from geophysical values due to earthquake excitation if the velocities are less than about 4200 ft/sec. Site response analyses of vertical excitation for the Lotung and Port Island sites were conducted using the estimated values of compressional-wave velocities based on Fig. 13. Values of soil damping used were the average values estimated from the two horizontal components of excitation, but limited to less than 10 percent. Input motions recorded at a depth of 17 m were used for the Lotung site and at a depth of 83 m for the Port Island site.

Comparisons of the response spectra (5-percent damped) of the calculated motions and the recorded motions at the ground surface and other depths for the Lotung site and the Port Island site generally show that the response spectra of the calculated motions agree reasonably well with the recorded motions, although not as well as using the backcalculated compressional-wave velocities from the Fourier spectral ratio analyses.

CONCLUSION AND RECOMMENDATIONS

The results of the analyses conducted as part of this study indicate that the compressional-wave velocities of the near-surface unsaturated soils (even below the groundwater table) may be as much as 40 to 60 percent less than values determined from geophysical measurements. For those soil layers that have high compressional-wave velocities (close to or higher than that of water), reduction in compressional-wave velocity due to earthquake excitation is expected to be small.

Response spectra of the computed motions generally are in reasonably good agreement with those of the recorded motions when the site response analysis utilized the compressional-wave velocities estimated using the recommended relationship between the backcalculated values and geophysical values and estimated values of soil damping from the horizontal excitation, limiting soil damping to 10 percent.

Based on the results of the study, the following recommendations are made regarding site response analyses of vertical excitation.

- For near-surface unsaturated soils having compressional-wave velocities less than 4200 ft/sec, reduce compressional-wave velocities to values in accordance with the relationship shown on Fig. 13. For saturated soils having high compressional-wave velocities (greater than about 4200 ft/sec), use the values determined by geophysical measurements.
- For soil damping, use average values estimated from site response analyses for horizontal components, but limited to not more than 10 percent.

APPENDIX

Allen, N.F., Richart, F.E., Jr., and Woods, R.D. (1980). "Fluid wave propagation in saturated and nearly saturated sands." *Journal of Geotechnical Engineering*, ASCE, Vol. 106, No. GT3.

Anderson, D.G. (1991). "Geotechnical synthesis report for Lotung large-scale seismic experiment." Electric Power Research Institute, Palo Alto, California.

Chang, C.-Y., Mok, C.M., and Power, M.S. (1991). "Analysis of ground response data at Lotung large-scale soil-structure interaction experiment site." Report NP-7306-SL, Electric Power Research Institute, Palo Alto, California.

Chang, C.-Y., Mok, C.M., Tang, Y.K., and Tang, H.T. (1994). "Analysis of seismic vertical motion using Lotung downhole array data." Proceedings, Fifth U.S. National Conference on Earthquake Engineering, Chicago, Illinois, July 10-14, 1994.

Chang, C.-Y., Mok, C.M., and Tang, H.T. (1996). "Inference of dynamic shear modulus from Lotung downhole data." *Journal of Geotechnical Engineering*,

ASCE, Vol. 122, No. 8, pp. 657-665.

Committee of Earthquake Observation and Research in the Kansai Area (CEORKA), Japan, (1995). "Ground Motion Data from Kobe Earthquake."

Elgamal, A.-W., Zeghal, M., and Para, E. (1996). "Liquefaction of reclaimed island in Kobe, Japan." *Journal of Geotechnical Engineering*, ASCE, Vol. 122, No. 1, January.

Iwasaki, Y. (1995). "Geological and geotechnical characteristics of Kobe area and strong ground motion records by 1995 Kobe earthquake, Tsuchi-Kiso." *Japanese Soc. of Soil Mech. and Foundations Engineering*, Vol. 43, No.6 (in Japanese).

Seed, H.B., and Idriss, I.M. (1970). "Soil modulus and damping factors for dynamic response analyses." Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley.

Vuceti, M., and Dobry, D. (1991). "Effects of soil plasticity on cyclic response." *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 1, January.

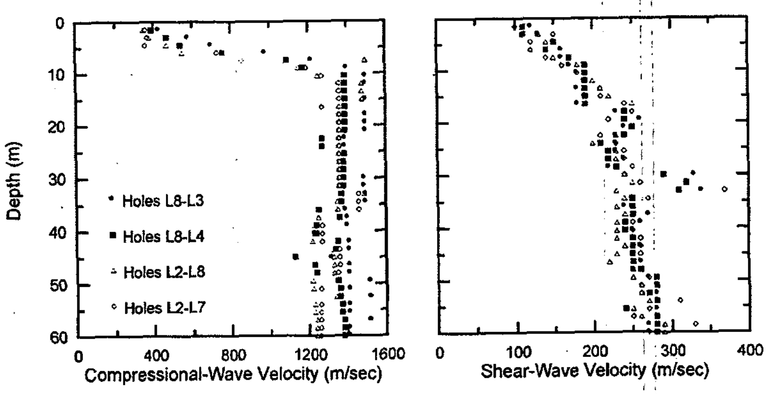


Figure 1. Shear- and compressional-wave velocity profiles at Lotung site

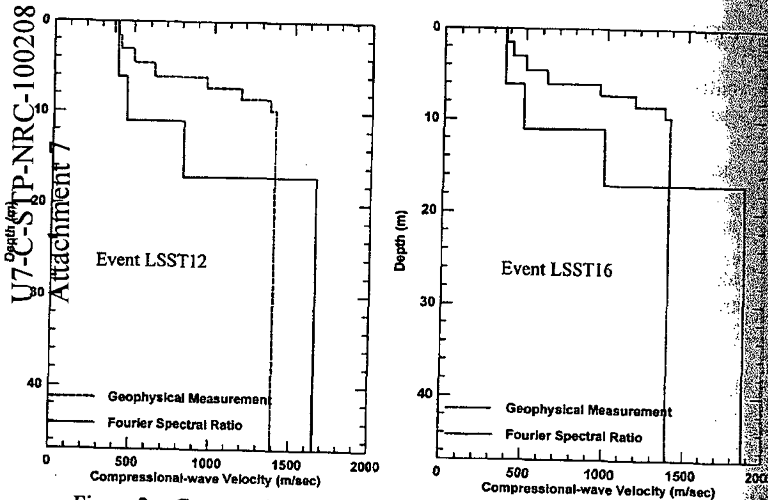


Figure 2. Compressional-wave velocities inferred from Fourier spectral ratios versus Geophysical measurements, Lotung site

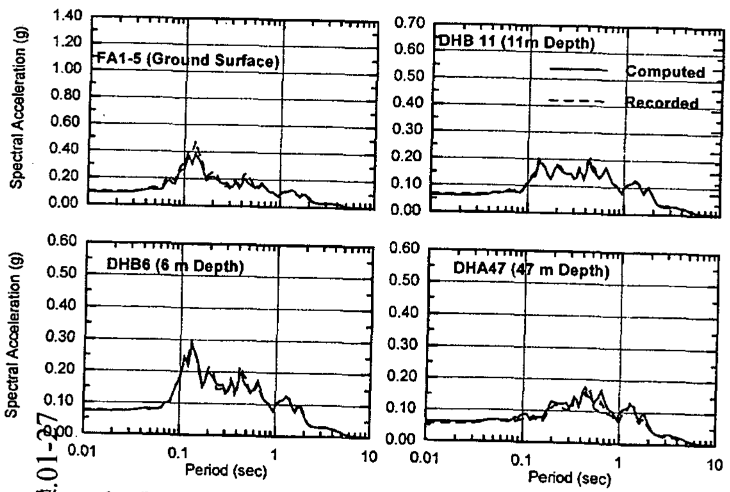


Figure 3. Response spectra (5% damped) of recorded and computed motions, case 1, Lotung site

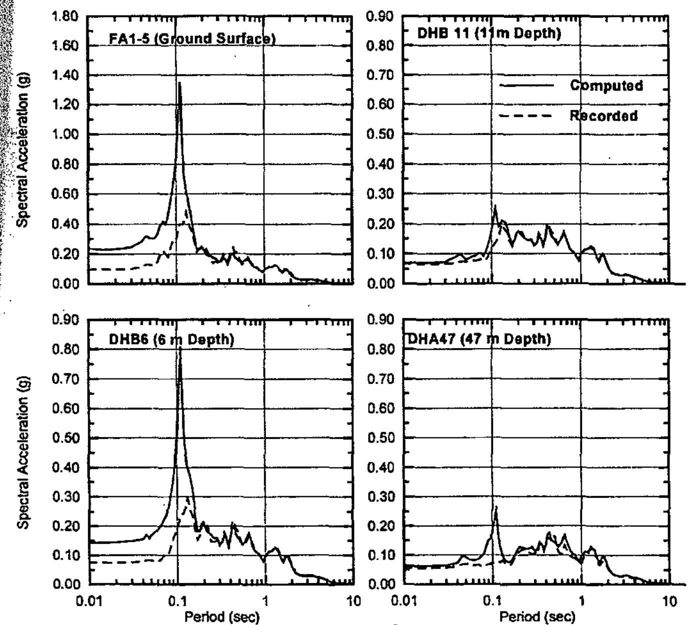


Figure 4. Response spectra (5% damped) of recorded and computed motions, case 2, Lotung site

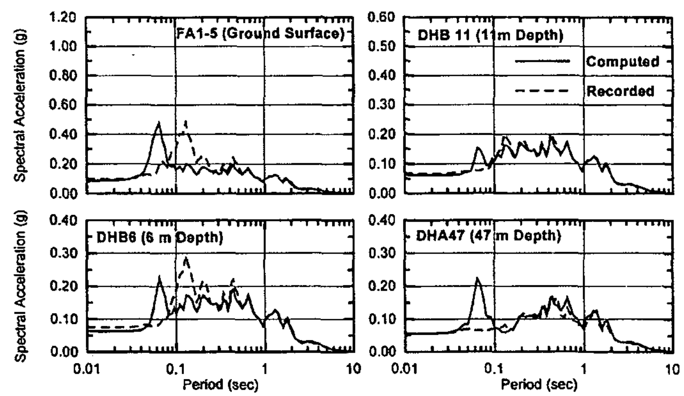


Figure 5. Response spectra (5% damped) of recorded and computed motions, case 3, Lotung site

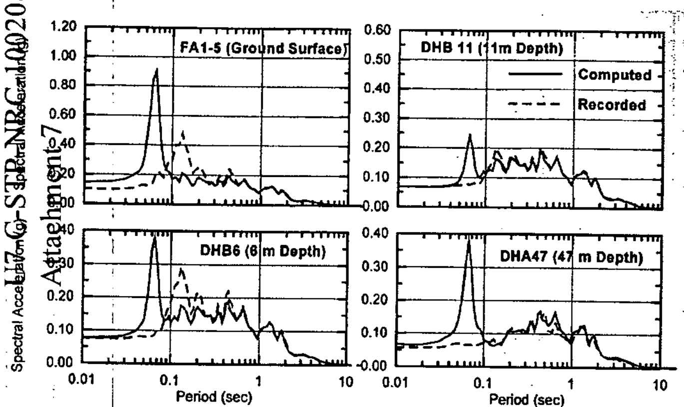


Figure 6. Response spectra (5% damped) of recorded and computed motions, case 4, Lotung site

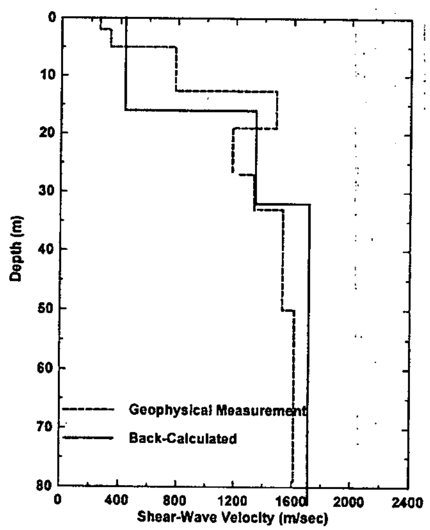


Figure 7. Compressional-wave velocities inferred from Fourier spectral ratios vs. geophysical measurements, Port Island site

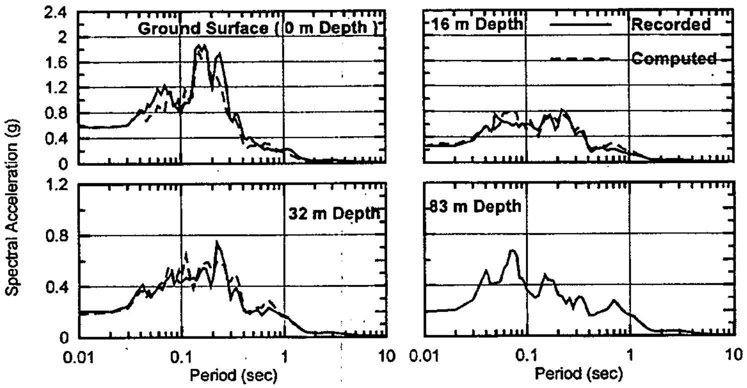


Figure 8. Response spectra (5% damped) of recorded and computed motions, case 1, Port Island site

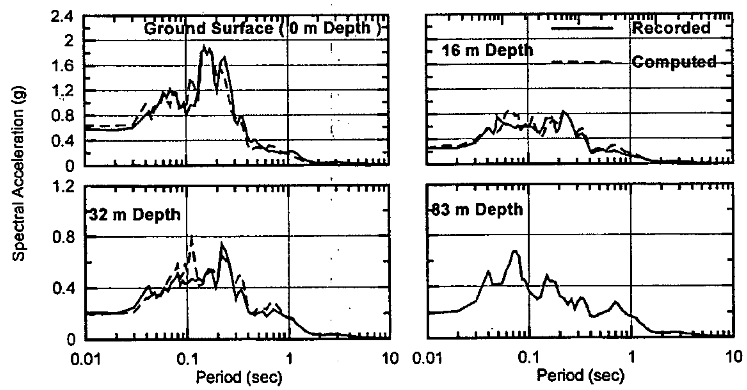


Figure 9. Response spectra (5% damped) of recorded and computed motions, case 2, Port Island site

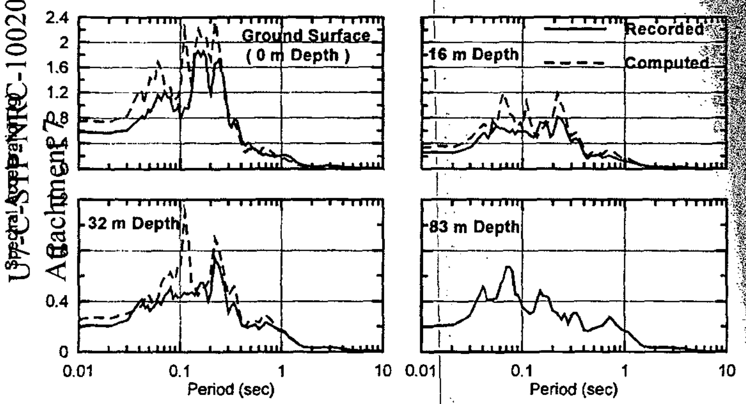


Figure 10. Response spectra (5% damped) of recorded and computed motions, case 3, Port Island site

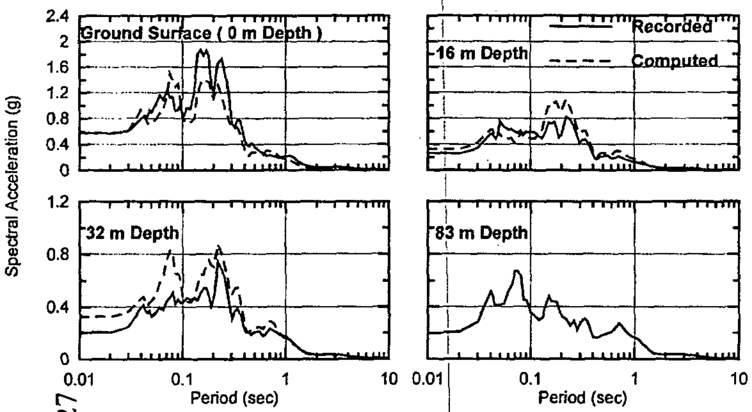


Figure 11. Response spectra (5% damped) of recorded and computed motions, case 4, Port Island site

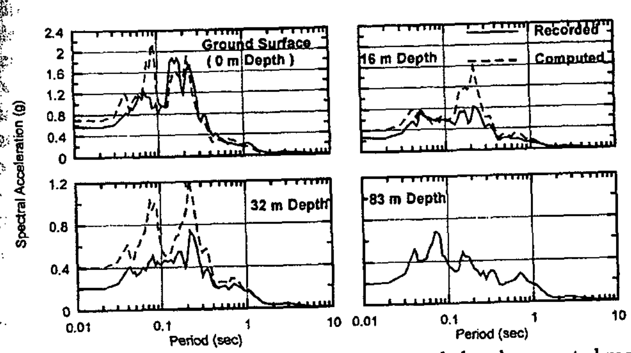


Figure 12. Response spectra (5% damped) of recorded and computed motions, case 5, Port Island site

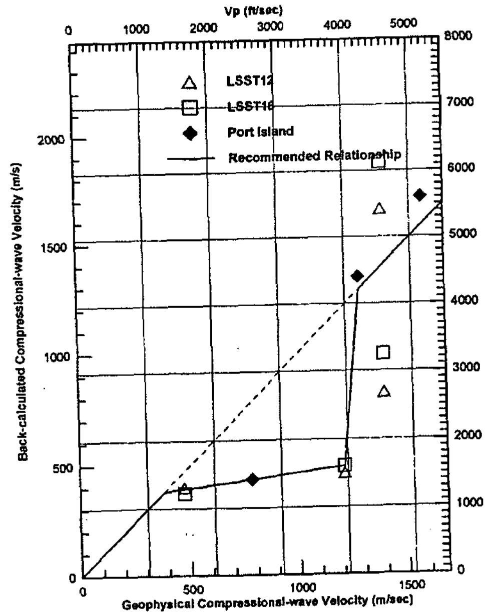


Figure 13. Recommended compression-wave velocities for site response analysis of vertical excitation

RAI 03.07.01-28

QUESTION:

Follow-up Question to RAI 03.07.01-20 (STP-NRC-100036)

In the response to Item 2a) of the RAI 03.07.01-20, the applicant has calculated the site-specific vertical and horizontal soil spring values for the STP soil conditions for the Control Building (CB) using drained Poisson's ratios of 0.15 to 0.30. The weighted soil spring values obtained for the STP best estimate, upper range, and lower range soil cases are shown in Table 03.07.01-20c, where they are compared against those estimated using the soil input from DCD, Section 3H.2.4.2.1. For the best estimate and upper range soil cases, the calculated site-specific soil spring values for the CB are the same or higher than those of the DCD; for the lower range soil case, the calculated spring constants are lower than those of the DCD.

To evaluate the impact of the lower spring constants calculated for the CB on the mat design, the applicant has performed a sensitivity analysis comparing the stresses in the CB base mat obtained using the site-specific lower range spring values versus those obtained using the DCD-derived soil spring constants. This analysis was performed for the total dead load of the structure with seismic moment applied about the x-axis (along East-West). Based on the results of this analysis, the applicant has stated that there is no significant difference in the mat stresses calculated using site specific and DCD spring values.

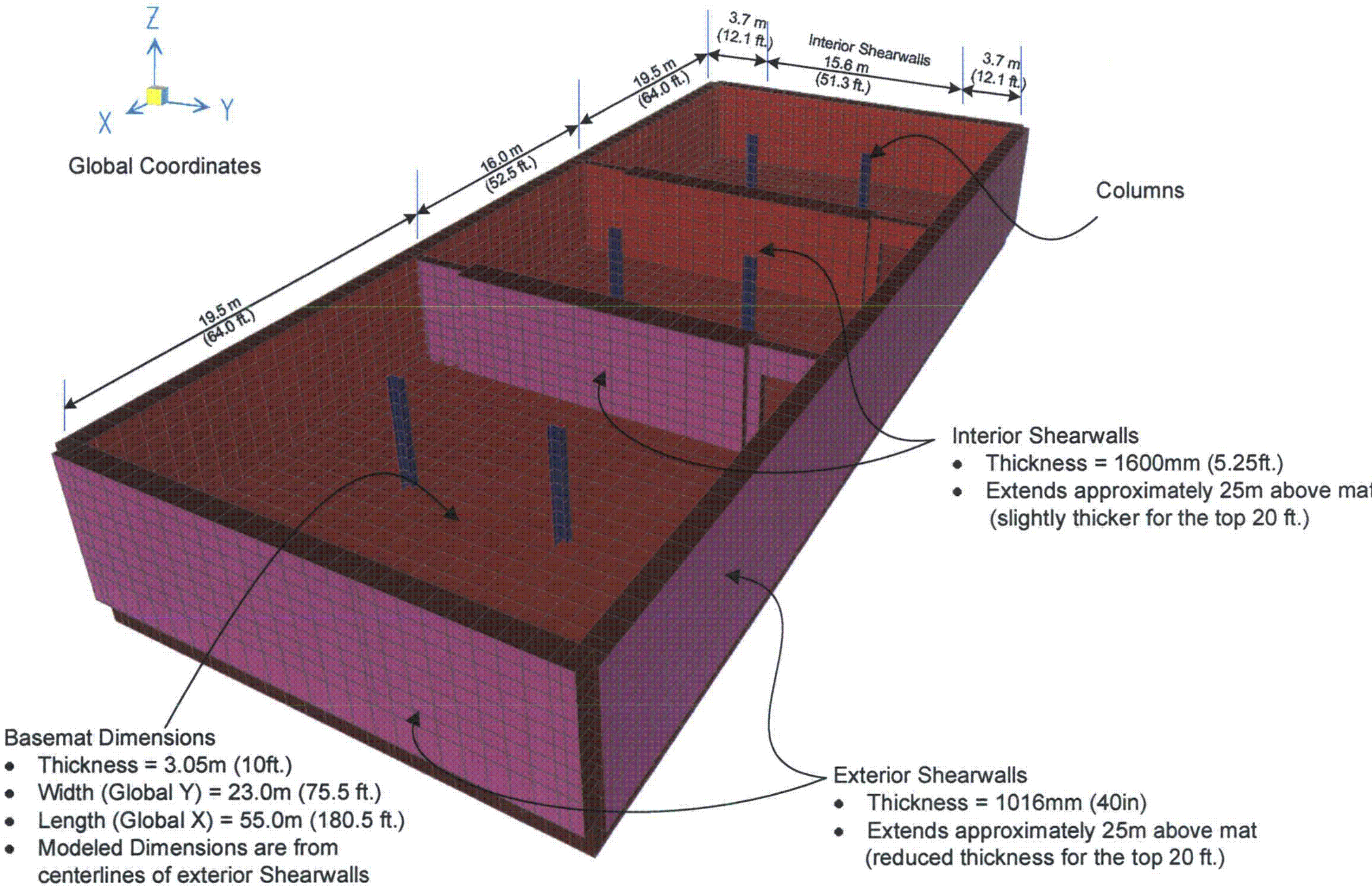
In evaluating the mat stress analysis results, it is noted that for the seismic load combination, the seismic moment has been applied about the x-axis (along East-West) in which the mat is expected to behave in a more rigid manner (with the results presented in Figures 03.07.01-20b through 03.07.01-20i). However, it is not clear whether the stress analysis of the CB mat foundation included the vertical seismic loads. Furthermore, the mat is expected to behave in a more flexible manner about the y-axis (North-South direction) as compared to the x-axis (East-West direction) (as the mat thickness/length ratio is larger in the y-direction as compared to the x-direction, and the two shear walls in the y-direction have no stiffening effect on the mat flexural behavior about the y-axis). As such, the applicant is requested to evaluate the mat stresses due to seismic moment acting about the y-axis. The applicant is also requested to clarify whether the vertical seismic loads were included in the sensitivity analysis, and if not what is the justification for not including the vertical seismic loads in the mat stress analyses. The staff needs this information to conclude that CB foundation mat on STP site will be bounded by the standard plant CB design.

RESPONSE:

In the sensitivity/parametric study presented in response to RAI 03.07.01-20 vertical excitation was not considered because it would not have any impact on the conclusion of the parametric study. In order to demonstrate that neither inclusion of vertical excitation nor consideration of moment about the Y-axis will have any impact on the conclusion of the parametric study presented in response to RAI 03.07.01-20, the parametric study was repeated as follows.

Figure 03.07.01-28.1 shows the layout of the mat and the shear walls of a structure with a very similar arrangement to that of the Control Building as described in the DCD. The model used for this parametric study is a three dimensional finite element model. This model was analyzed eight times for the total dead load of the structure, vertical excitation (up or down) along with significant seismic moment about either the X-axis (along East-West) or the Y-axis (along North-South), once with DCD best estimate spring constants and the second time with lower bound site-specific spring constants. Figures 03-07-01-28.2 through 03-07-01-28.33 present contour plots of the resulting out-of-plane moments and shears. Comparison of the resulting out-of-plane moments and shears from these figures show that there is no significant change in mat design forces.

No additional COLA revision is required as a result of this response.



3-D View of Parametric Study Model

- Cross Section up to 1st Level above Mat for clarity.
- Non Shearwalls not shown.

Figure 03.07.01-28.1

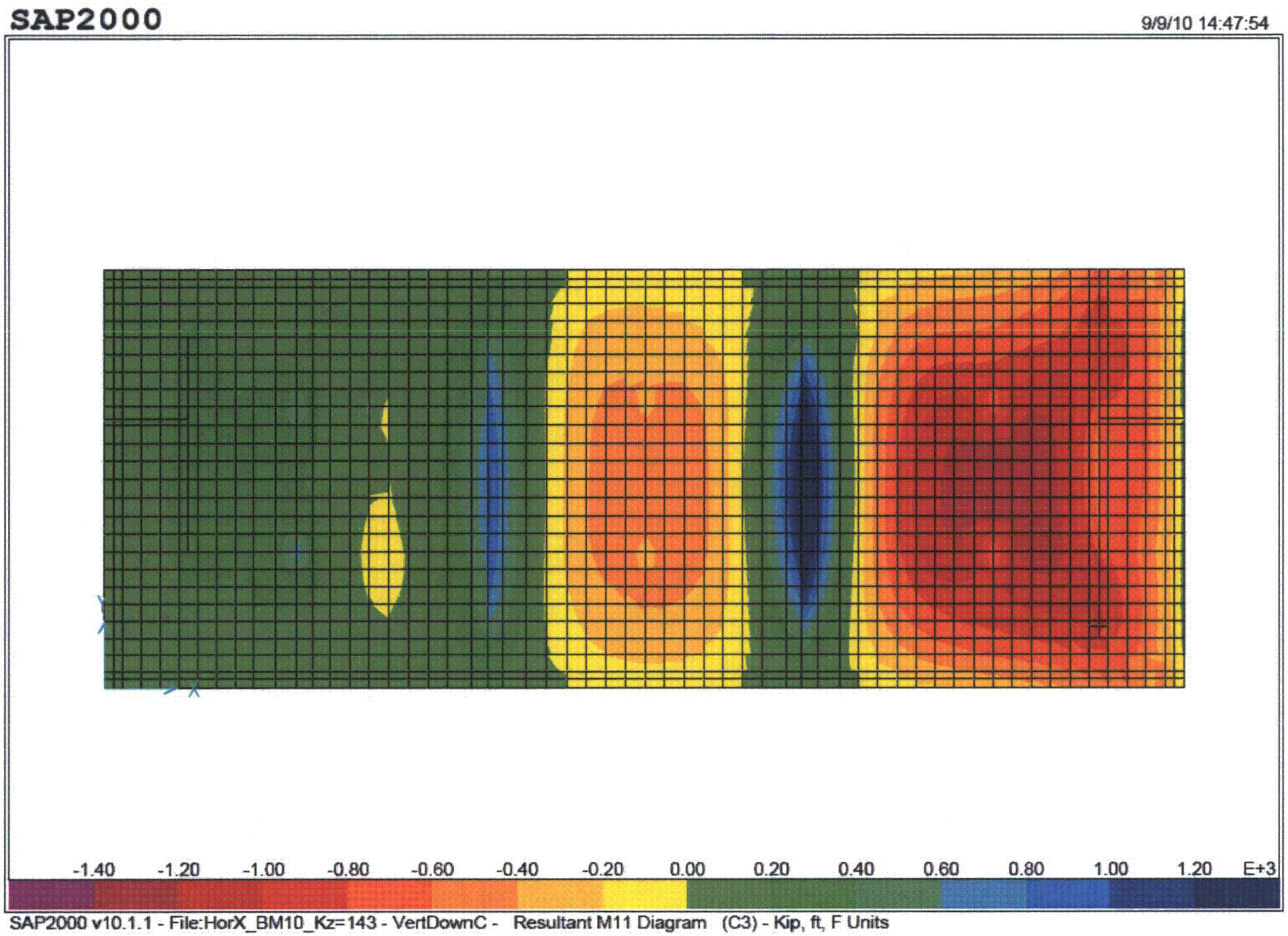


Figure 03.07.01-28.2: Resultant Out-of-Plane Moment M11 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_y

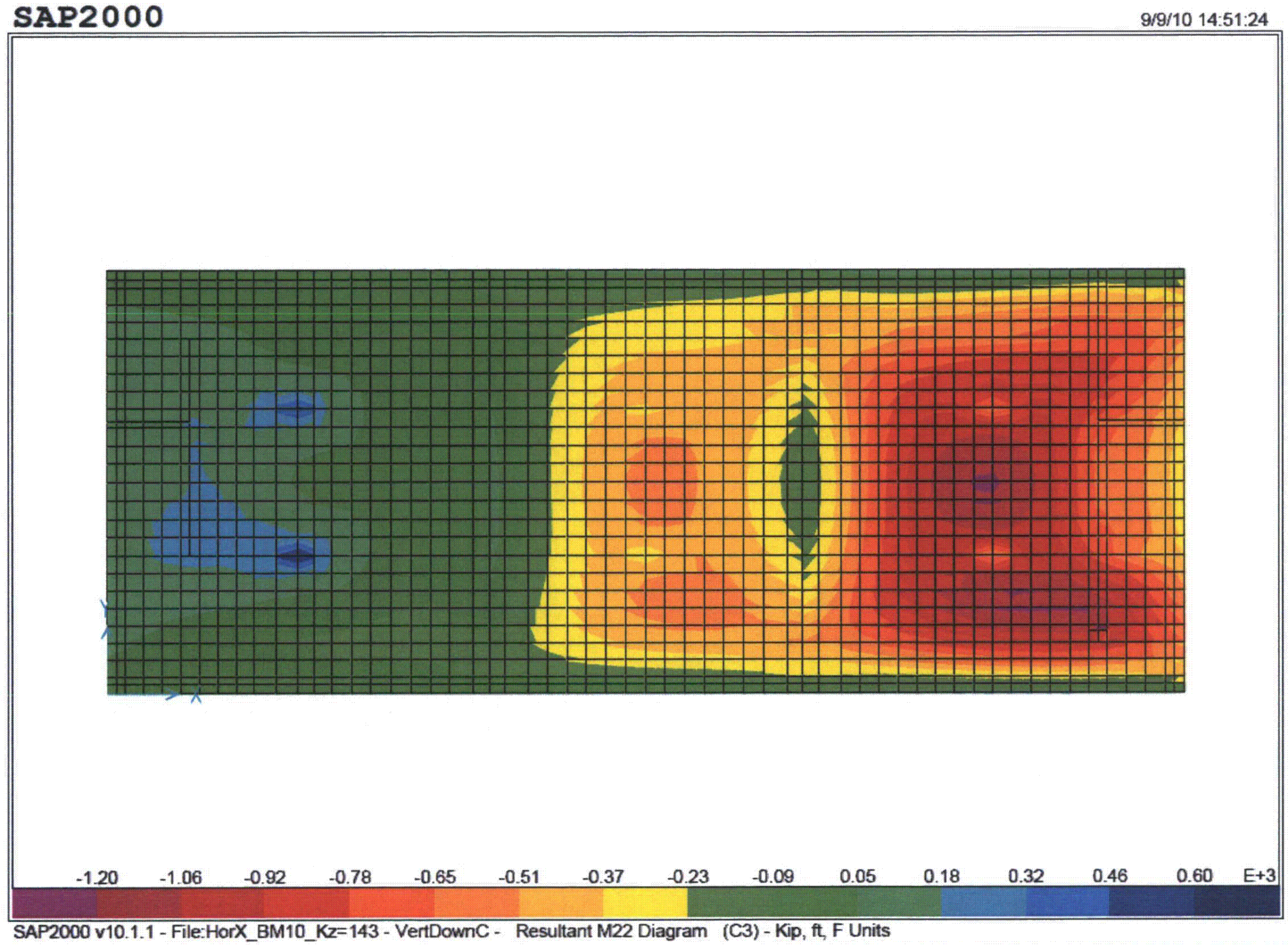


Figure 03.07.01-28.3: Resultant Out-of-Plane Moment M22 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_y

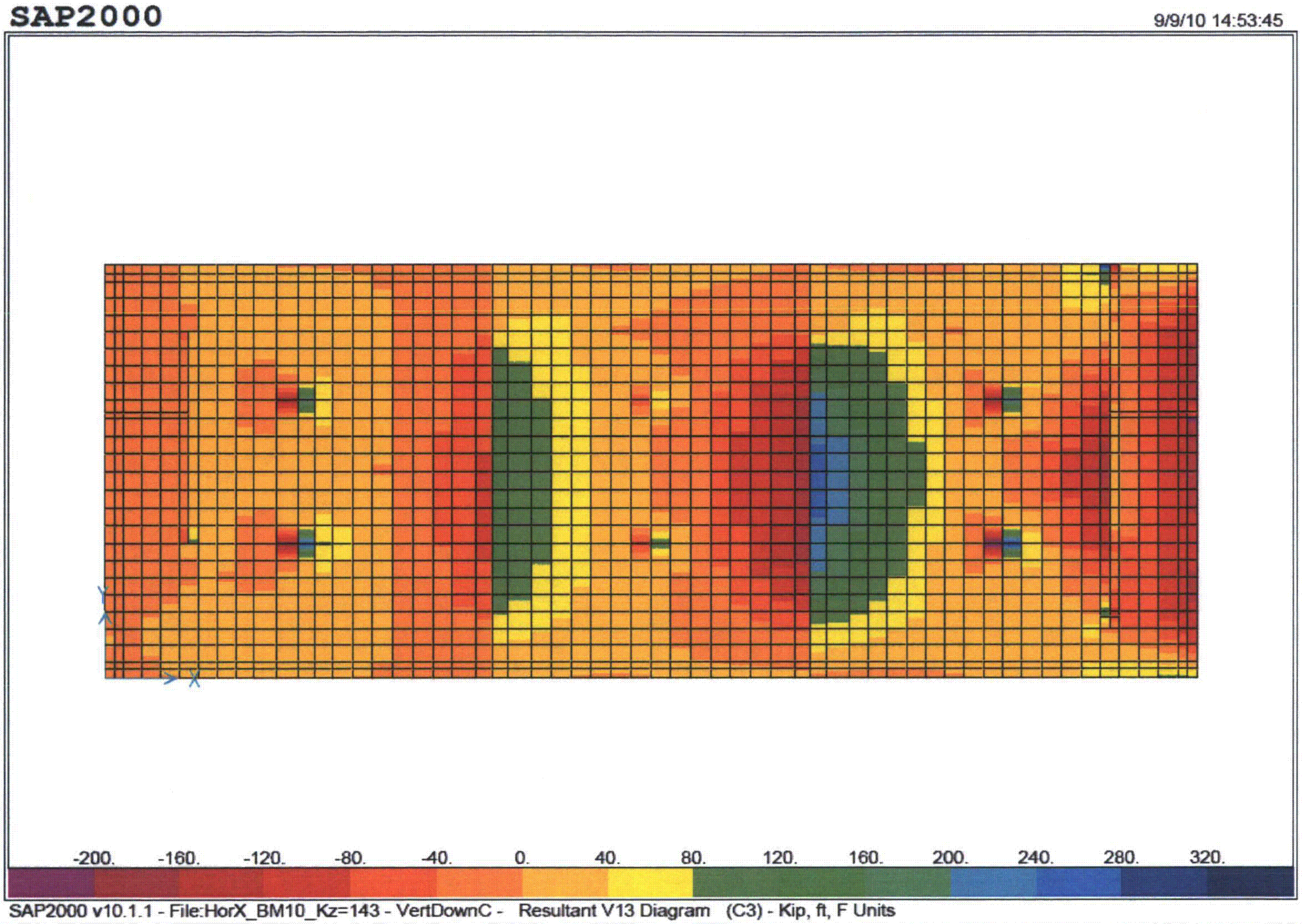


Figure 03.07.01-28.4: Resultant Out-of-Plane Shear V13 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_y

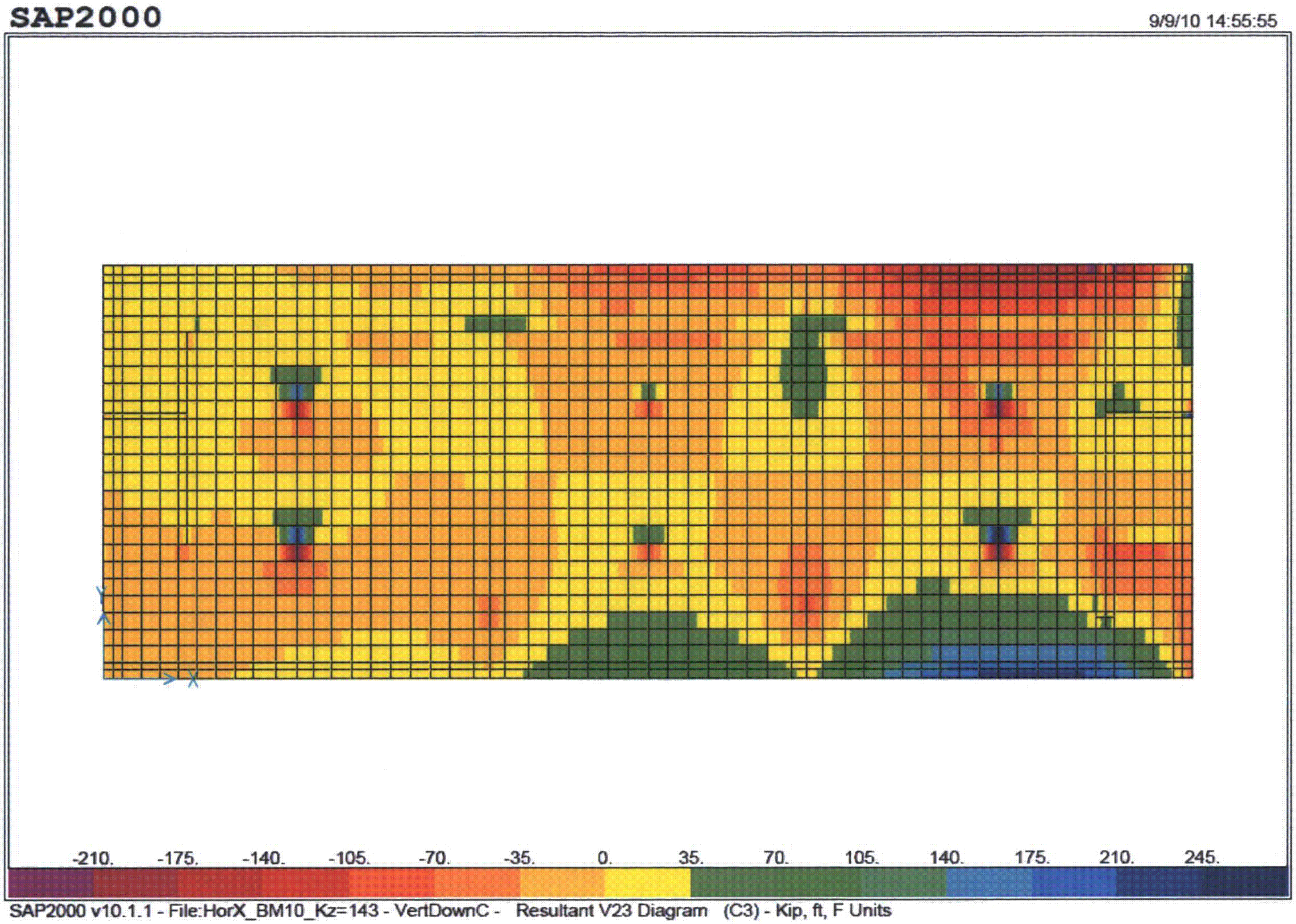


Figure 03.07.01-28.5: Resultant Out-of-Plane Shear V23 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_y

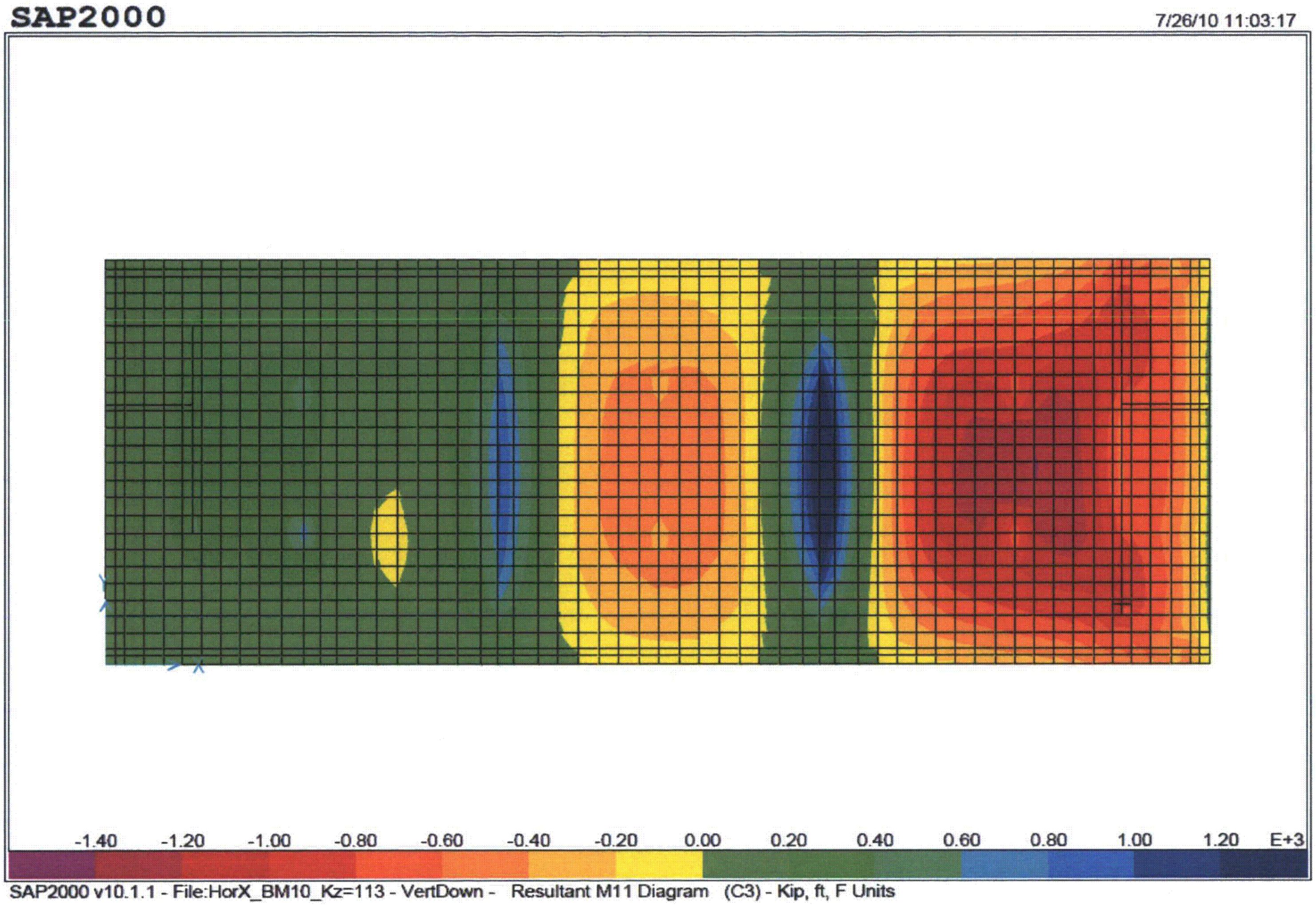


Figure 03.07.01-28.6: Resultant Out-of-Plane Moment M11 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_y

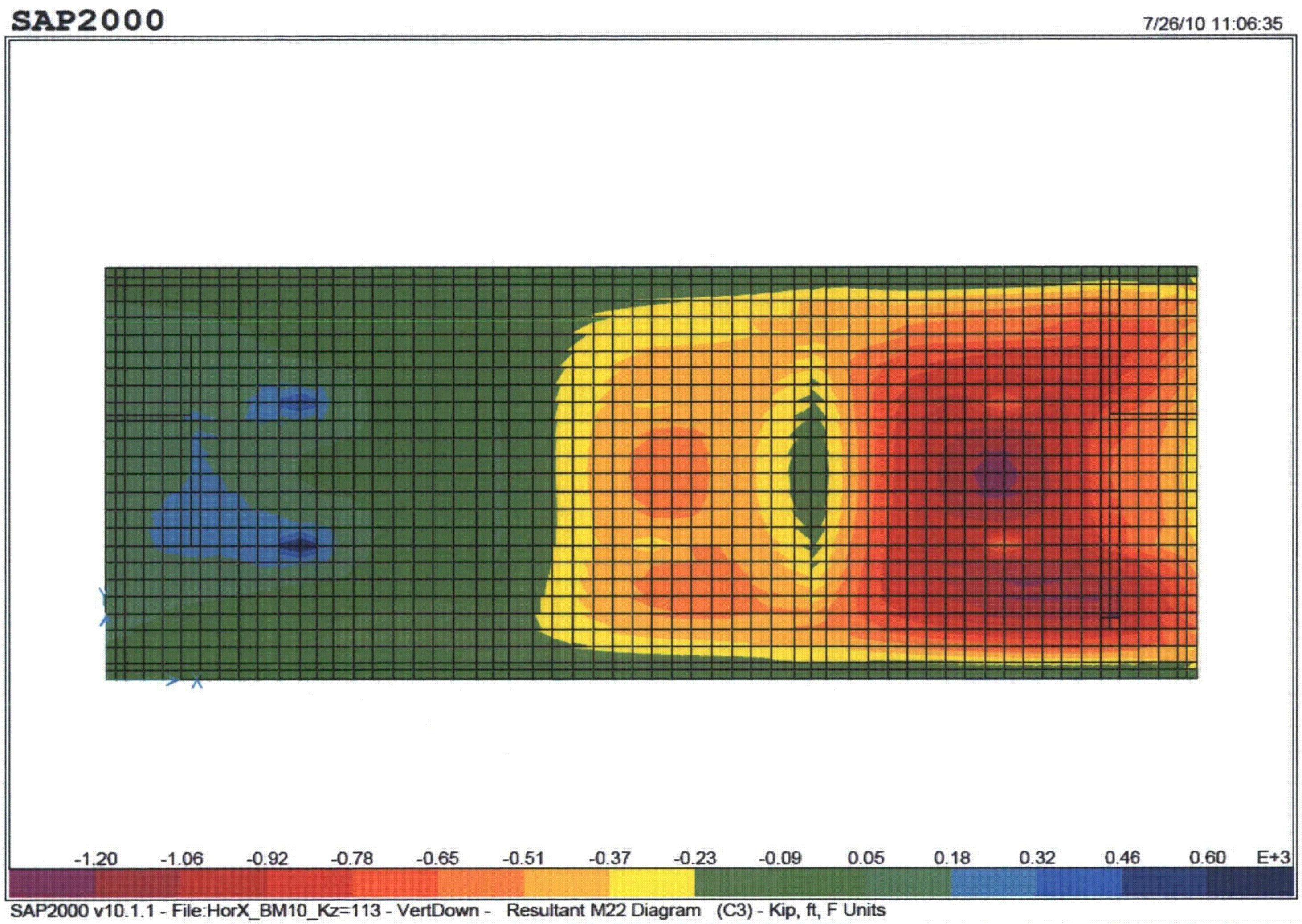


Figure 03.07.01-28.7: Resultant Out-of-Plane Moment M22 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_y

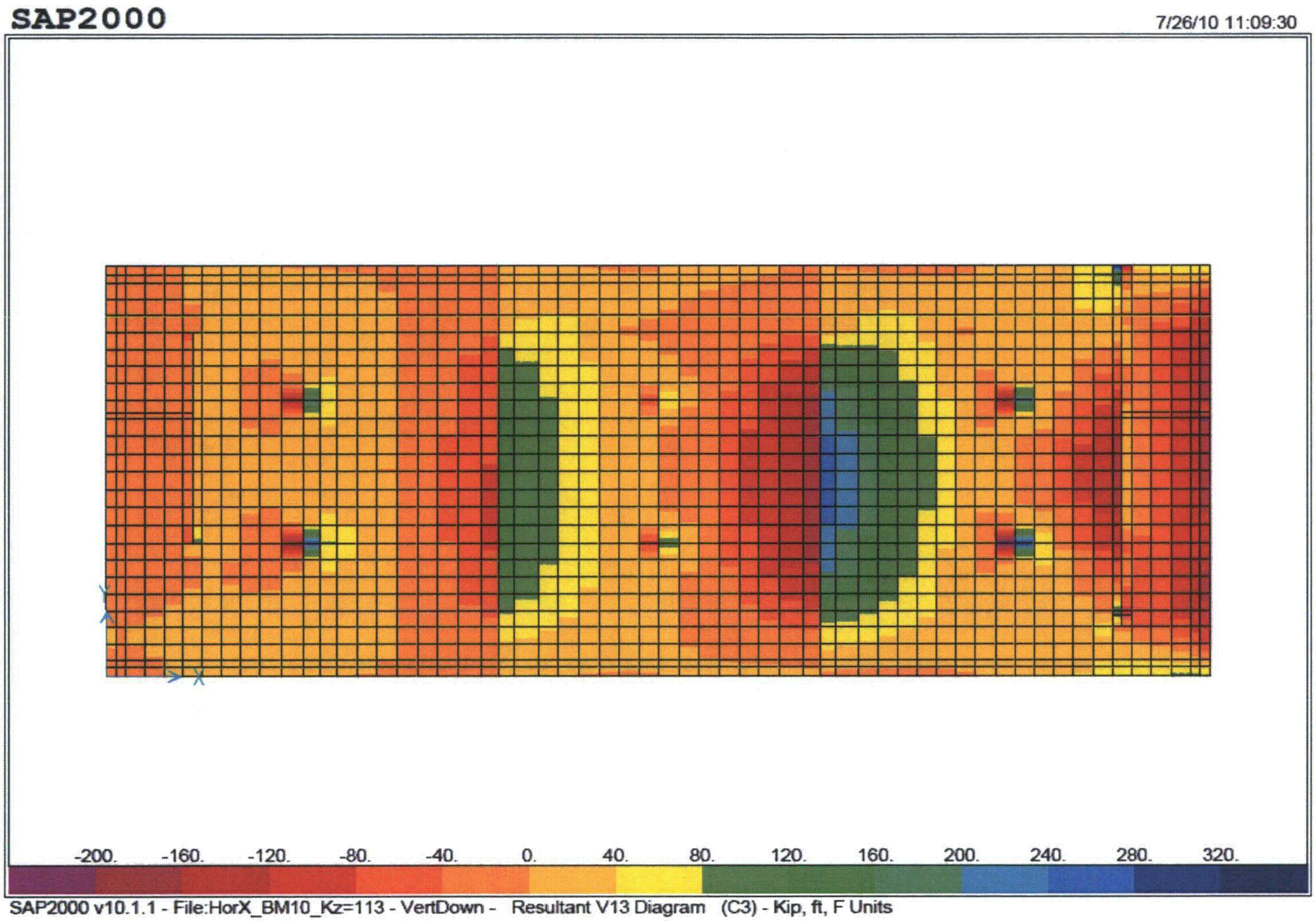


Figure 03.07.01-28.8: Resultant Out-of-Plane Shear V13 Diagram
(Using Lower Range Site-Specific Spring Constants)
Dead Load, Vertical Excitation Down, and Moment M_y

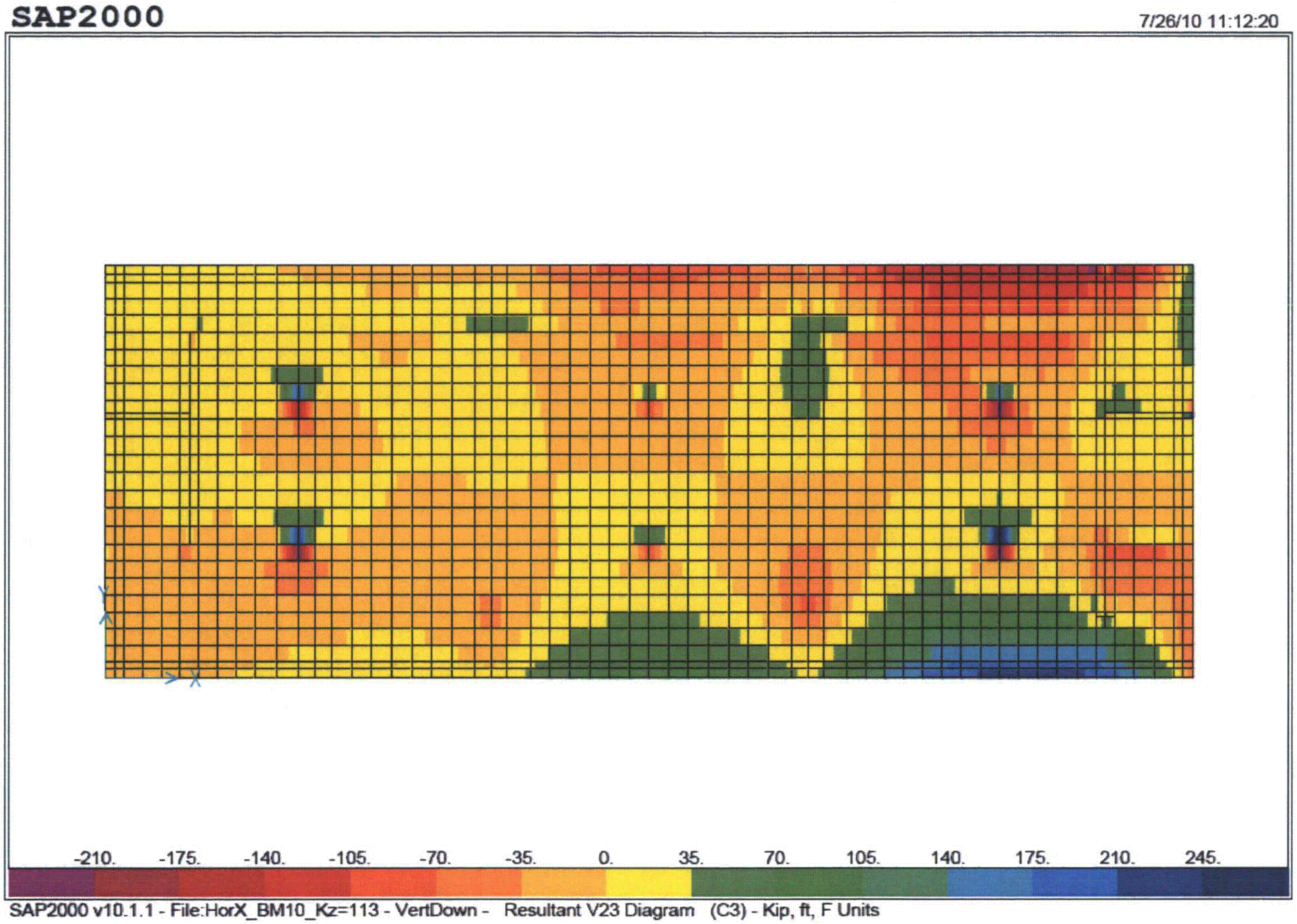


Figure 03.07.01-28.9: Resultant Out-of-Plane Shear V23 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_y

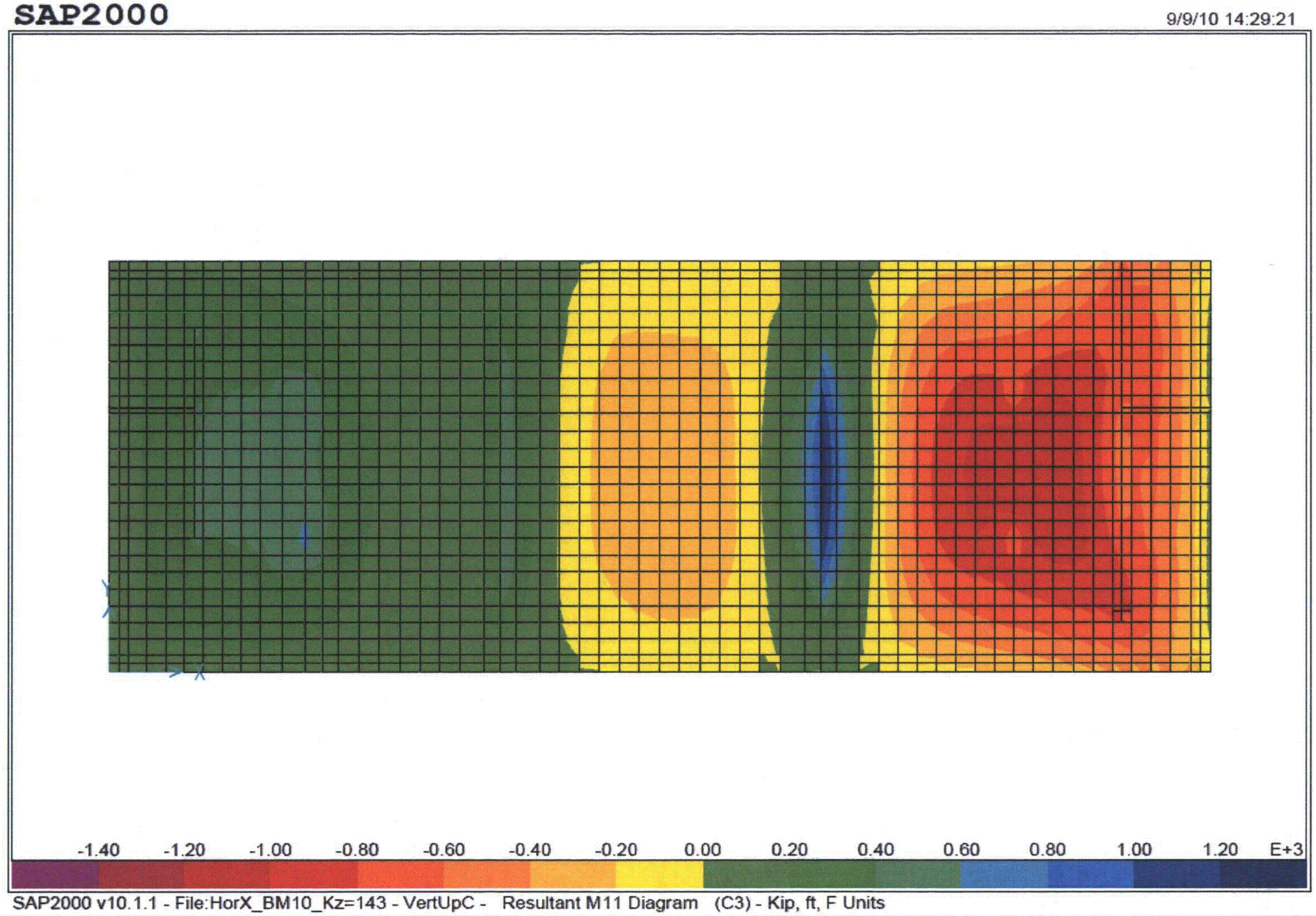


Figure 03.07.01-28.10: Resultant Out-of-Plane Moment M11 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

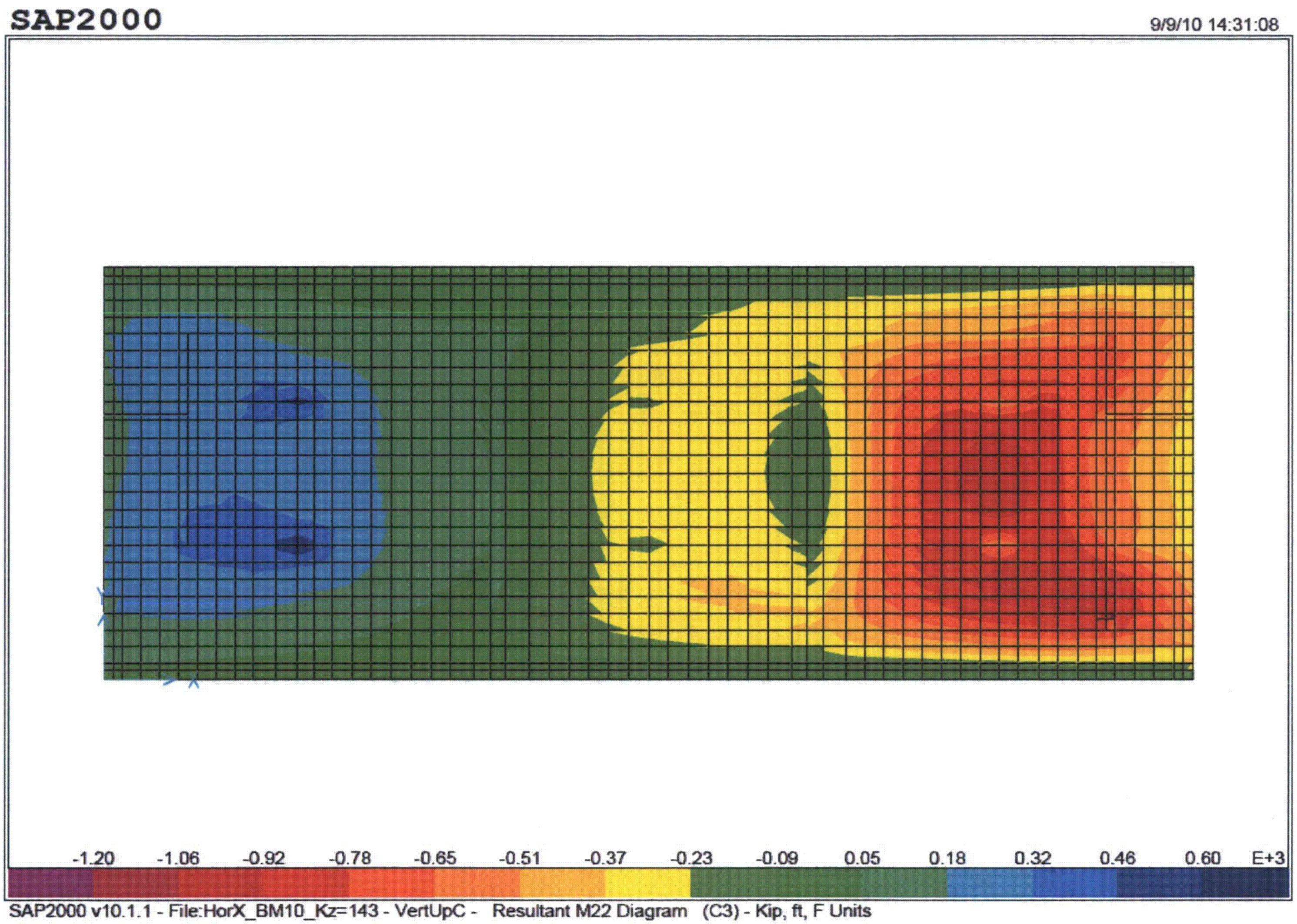


Figure 03.07.01-28.11: Resultant Out-of-Plane Moment M22 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

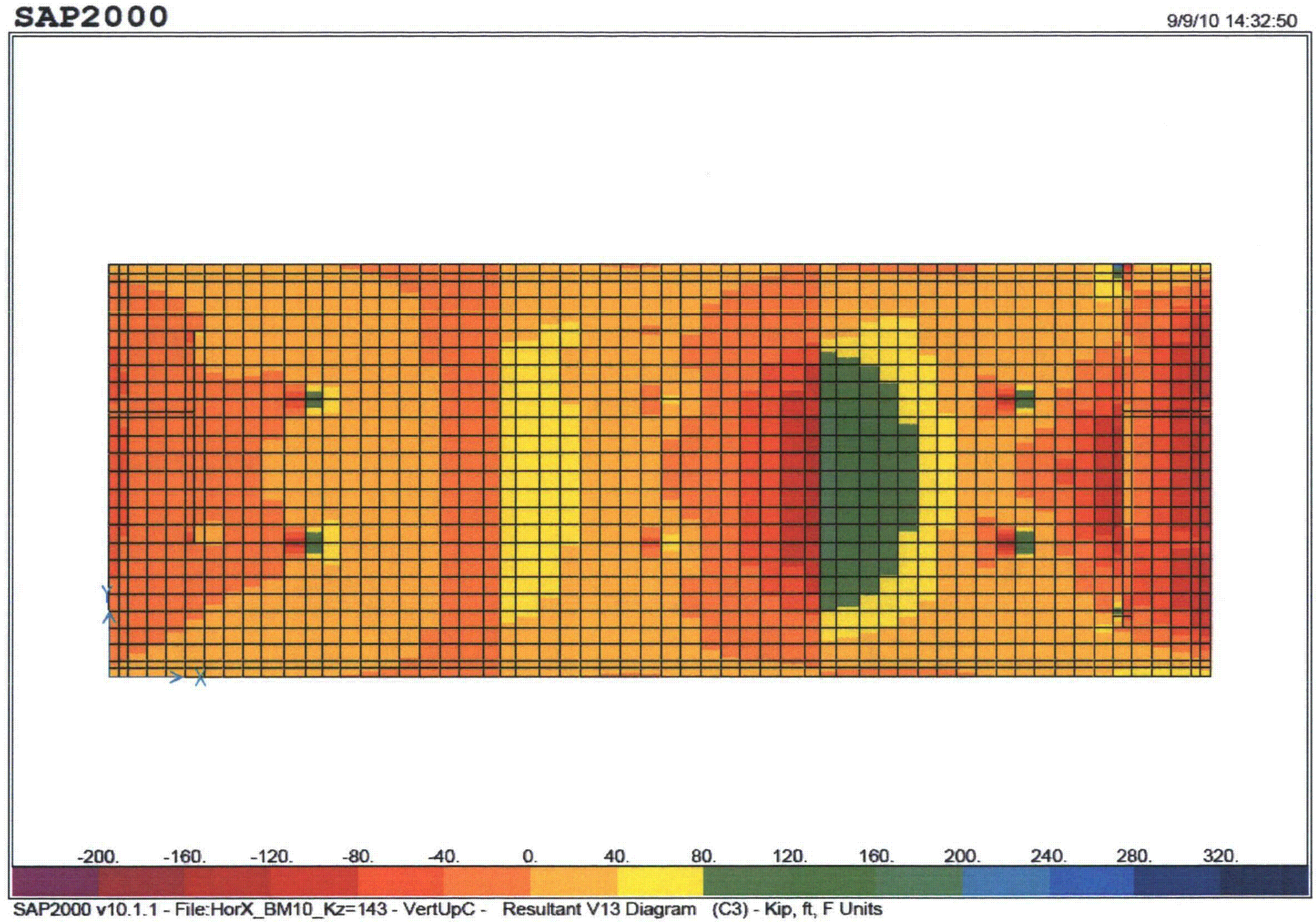


Figure 03.07.01-28.12: Resultant Out-of-Plane Shear V13 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

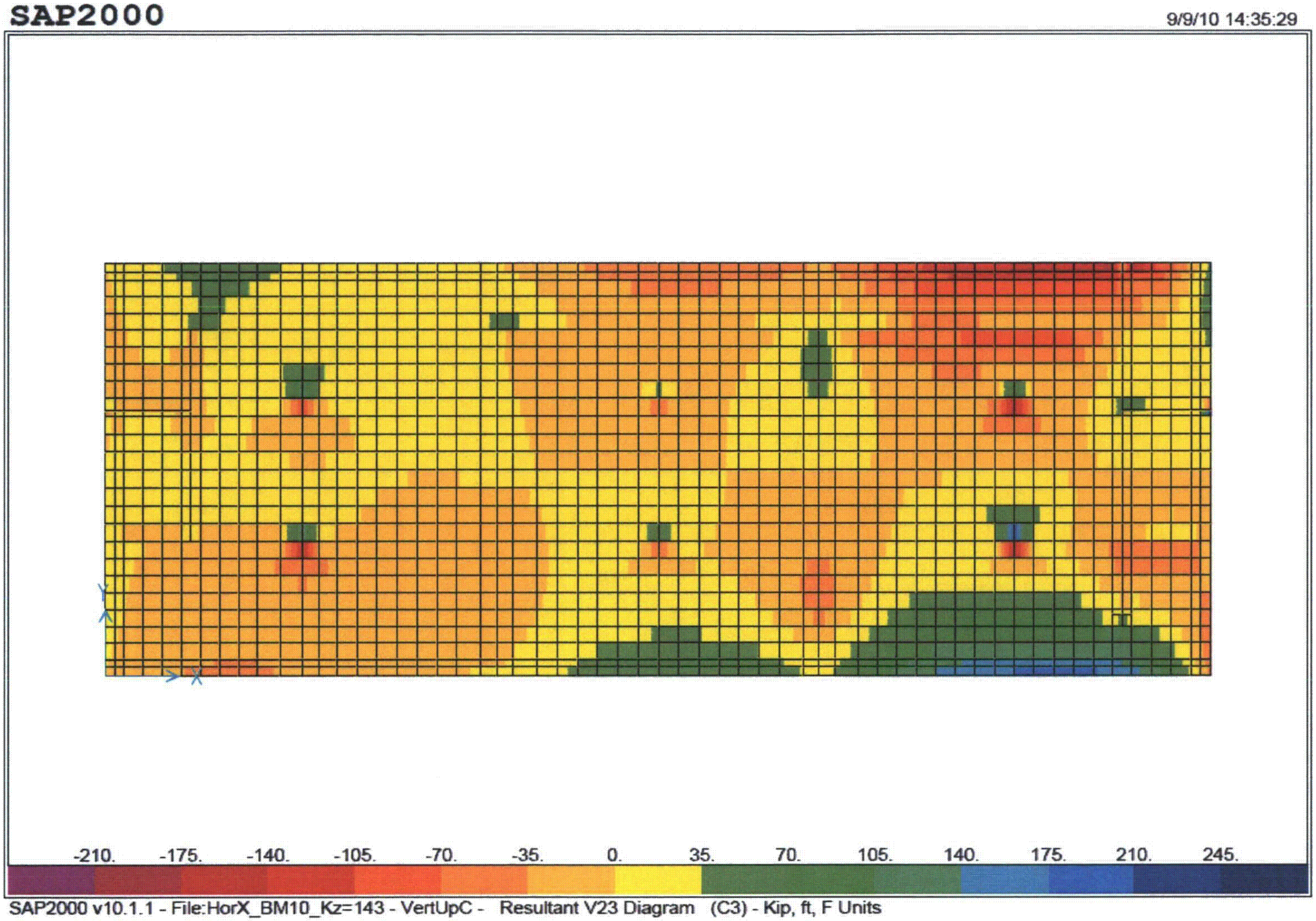


Figure 03.07.01-28.13: Resultant Out-of-Plane Shear V23 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

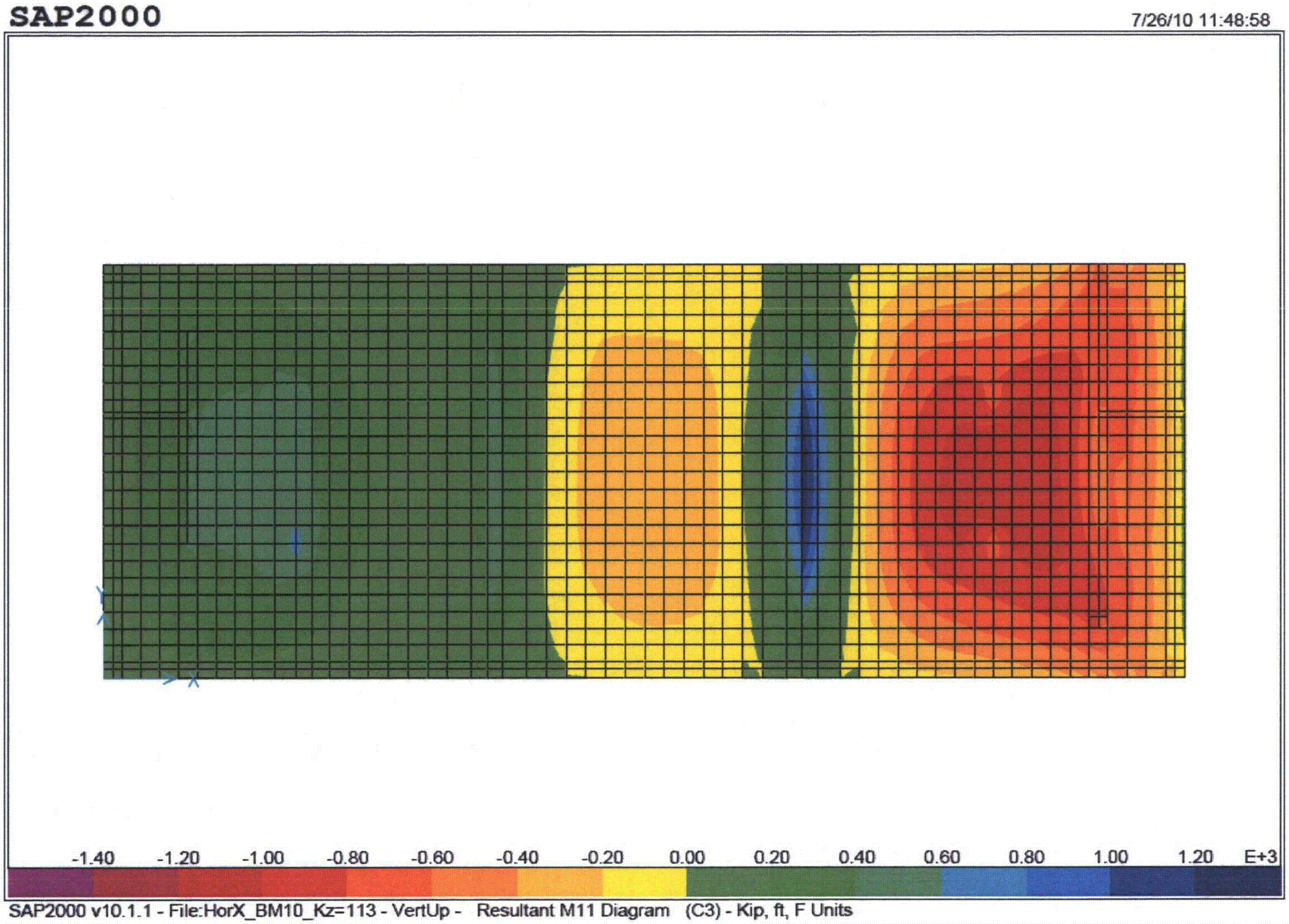


Figure 03.07.01-28.14: Resultant Out-of-Plane Moment M11 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

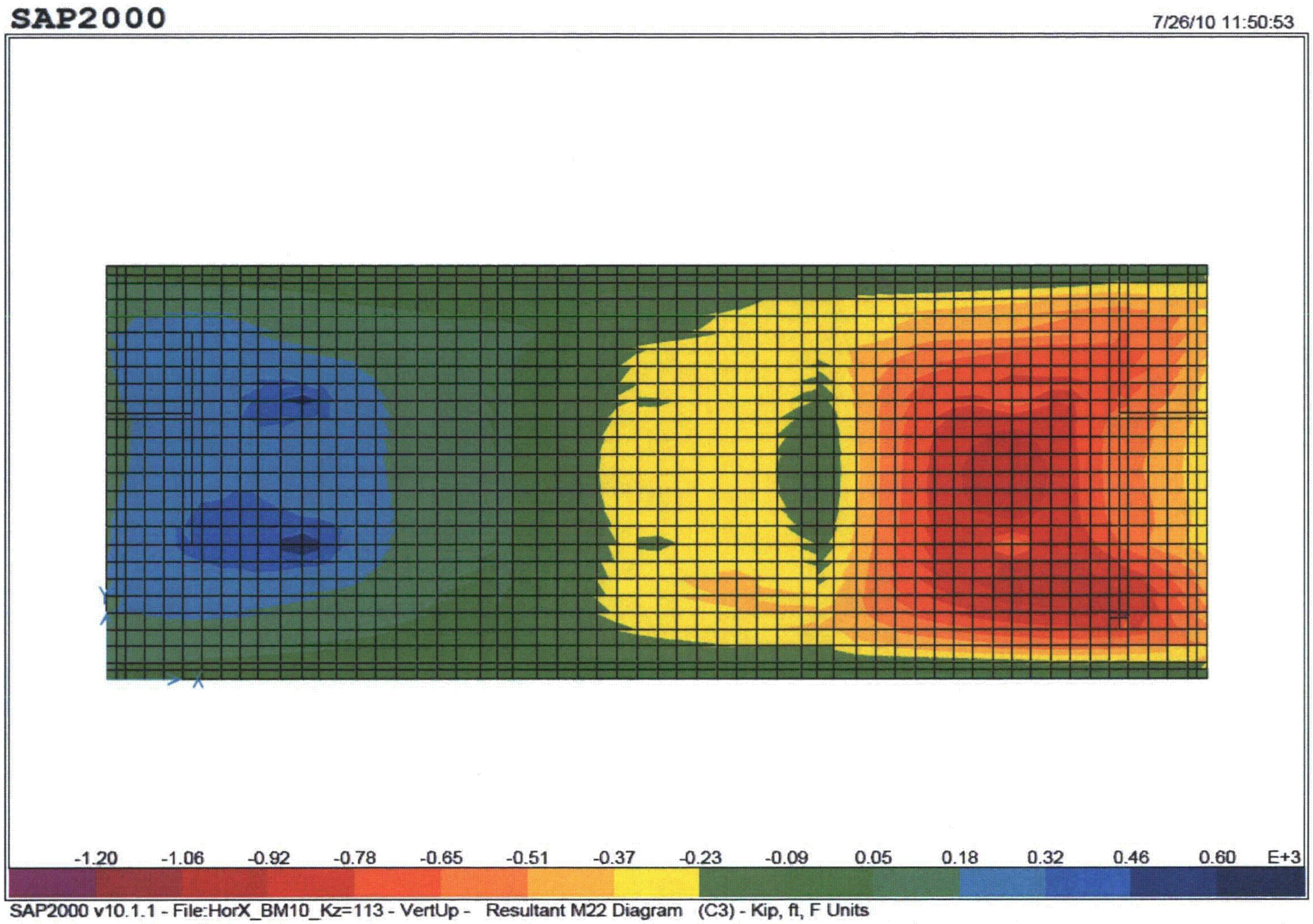


Figure 03.07.01-28.15: Resultant Out-of-Plane Moment M22 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

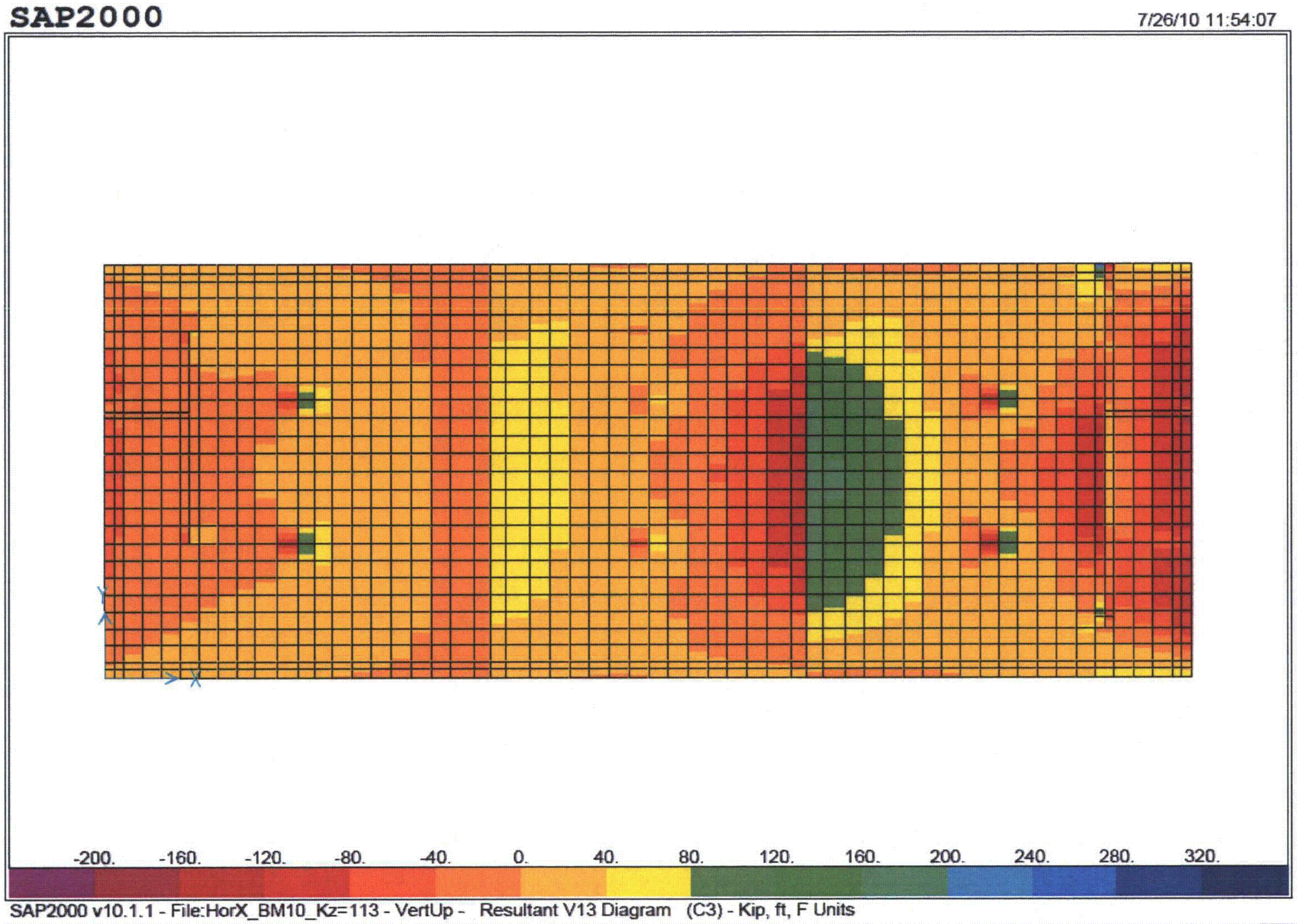


Figure 03.07.01-28.16: Resultant Out-of-Plane Shear V13 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

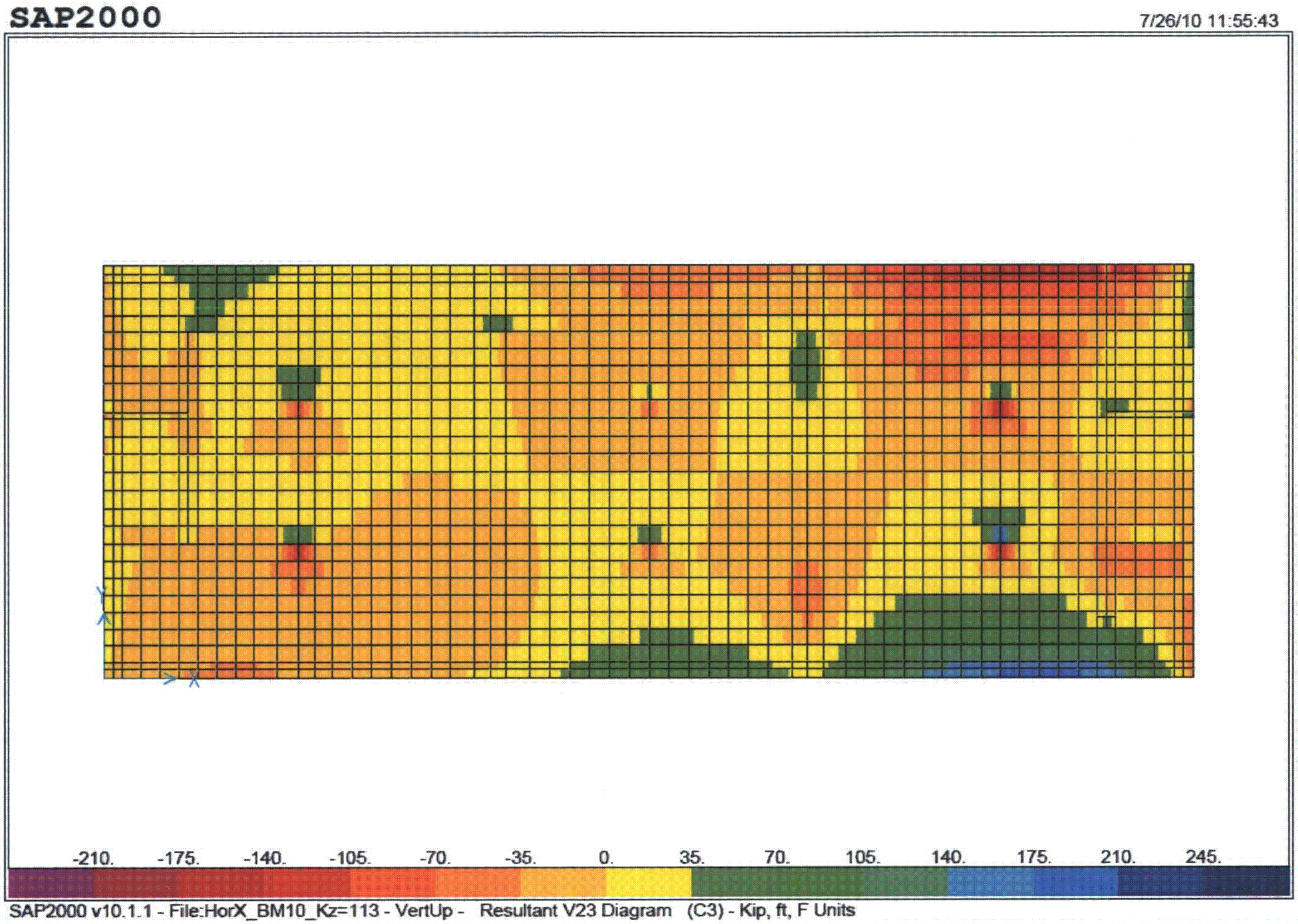


Figure 03.07.01-28.17: Resultant Out-of-Plane Shear V23 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_y

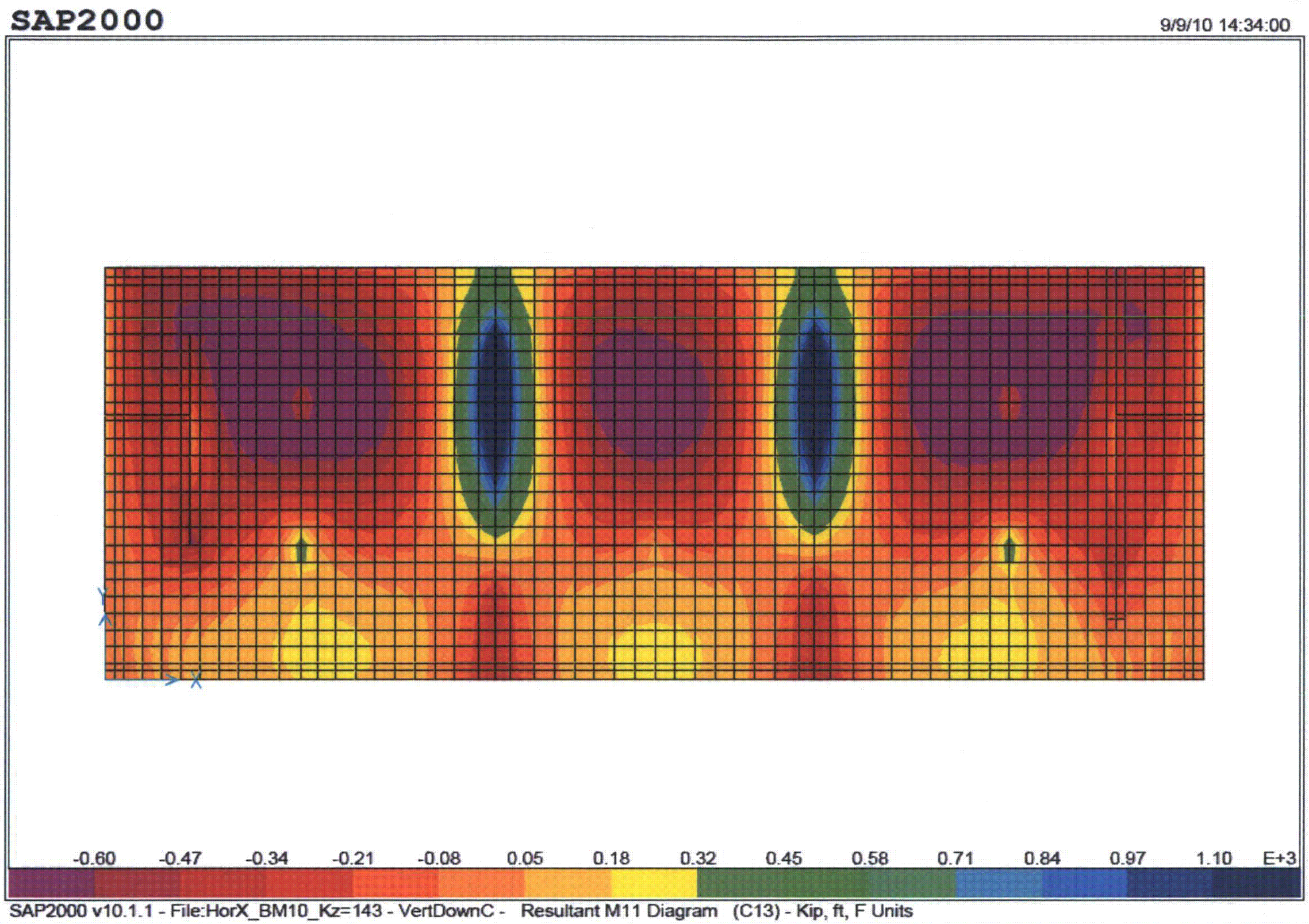


Figure 03.07.01-28.18: Resultant Out-of-Plane Moment M11 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

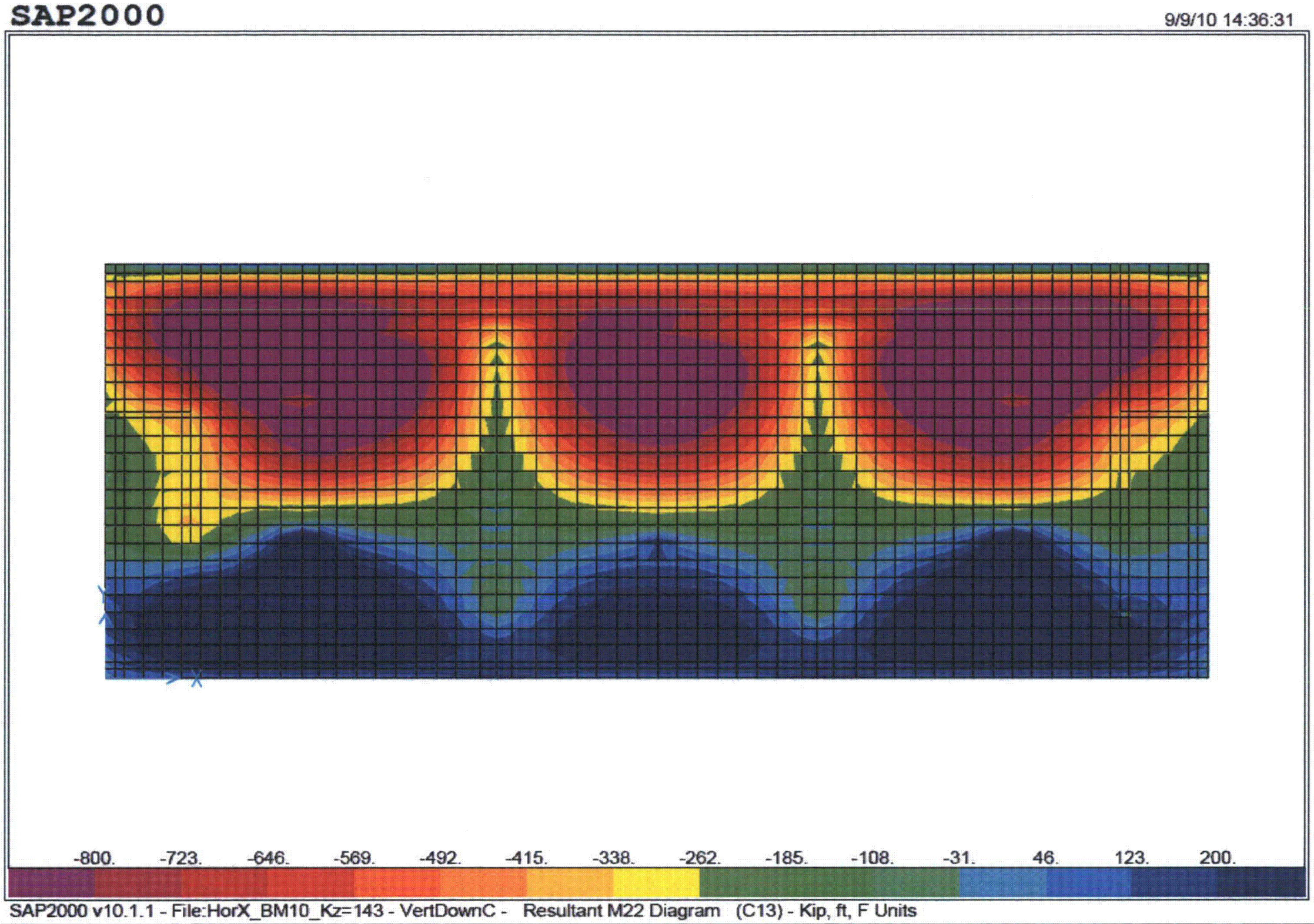


Figure 03.07.01-28.19: Resultant Out-of-Plane Moment M22 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

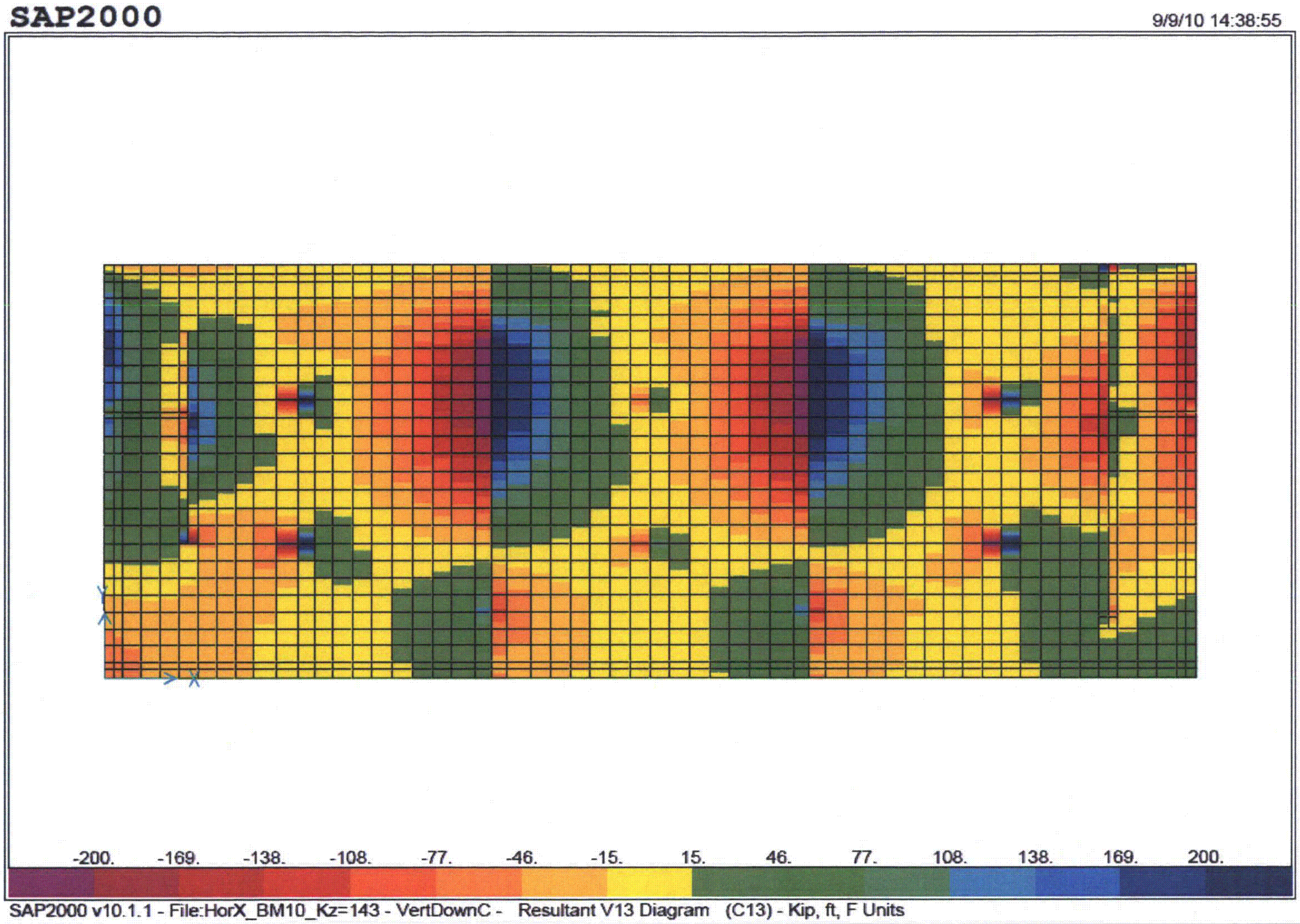


Figure 03.07.01-28.20: Resultant Out-of-Plane Shear V13 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

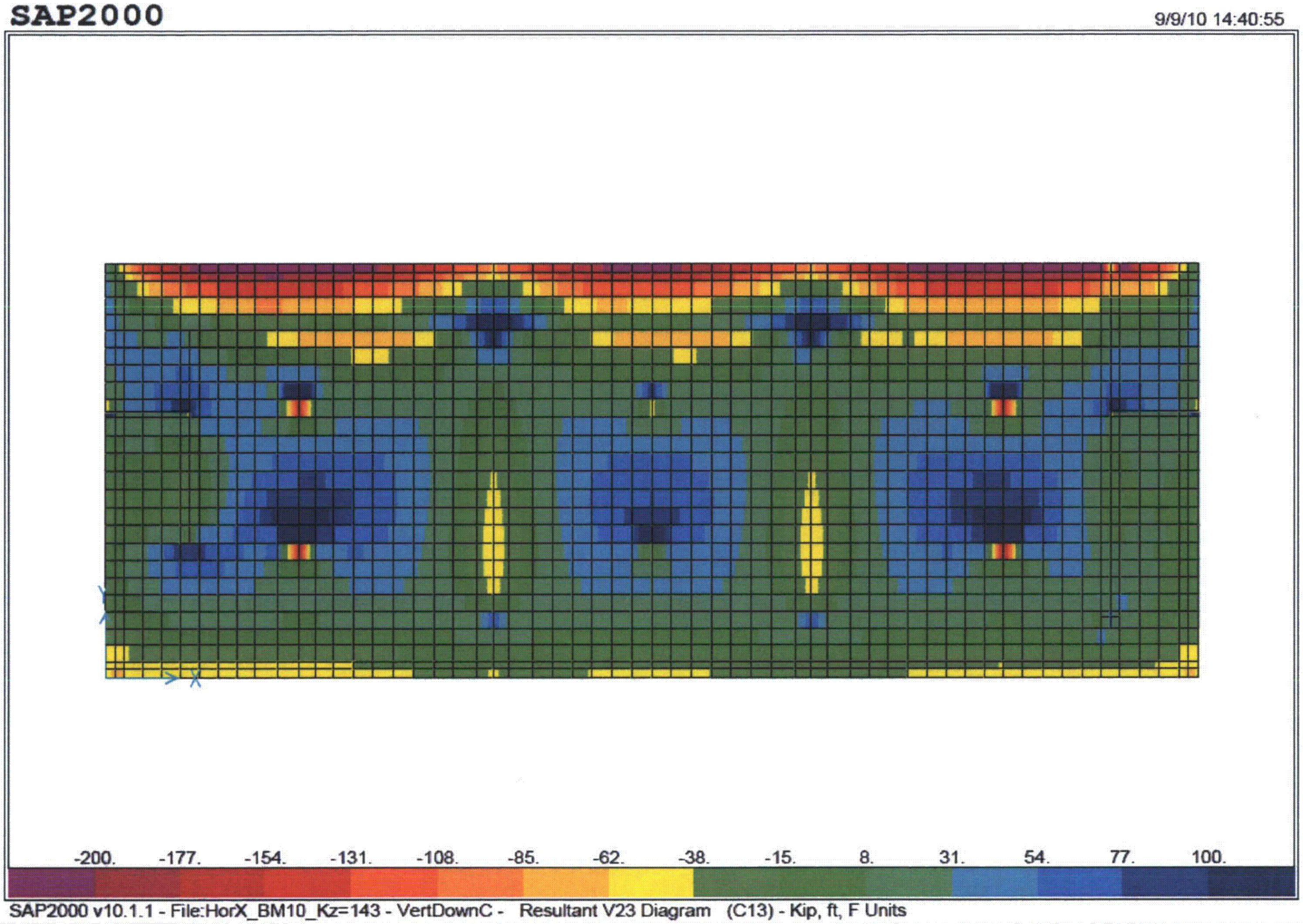


Figure 03.07.01-28.21: Resultant Out-of-Plane Shear V23 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

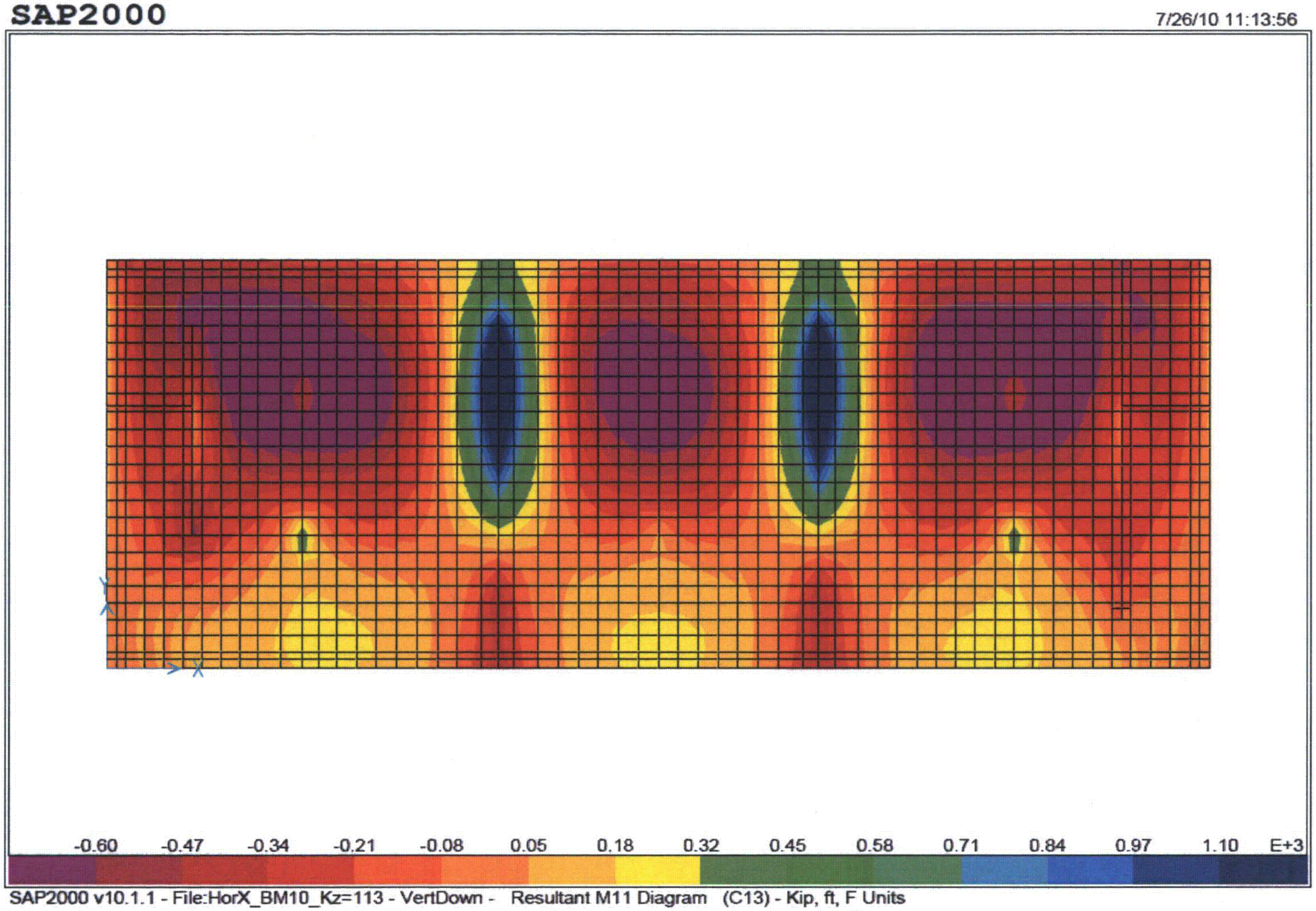


Figure 03.07.01-28.22: Resultant Out-of-Plane Moment M11 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

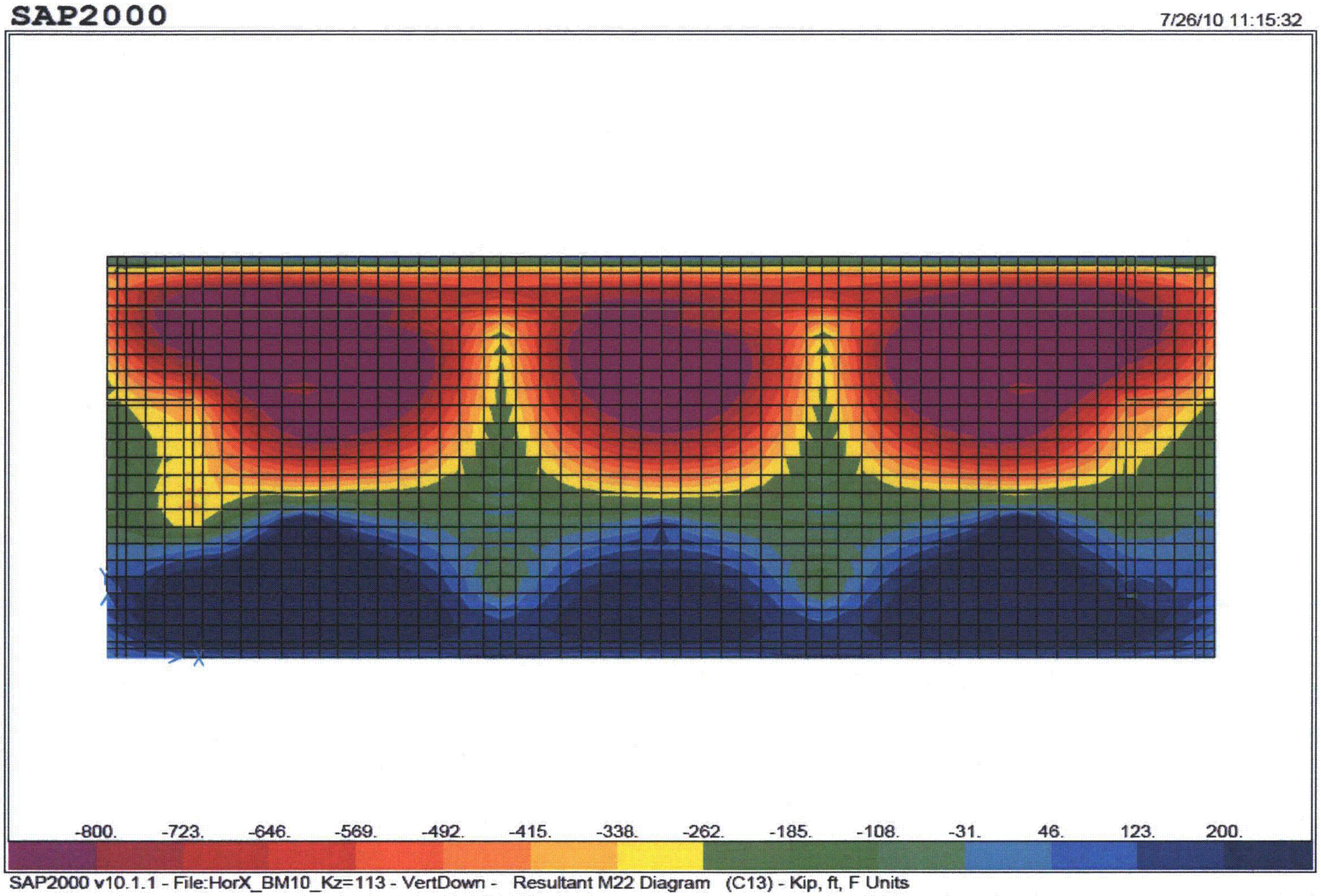


Figure 03.07.01-28.23: Resultant Out-of-Plane Moment M22 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

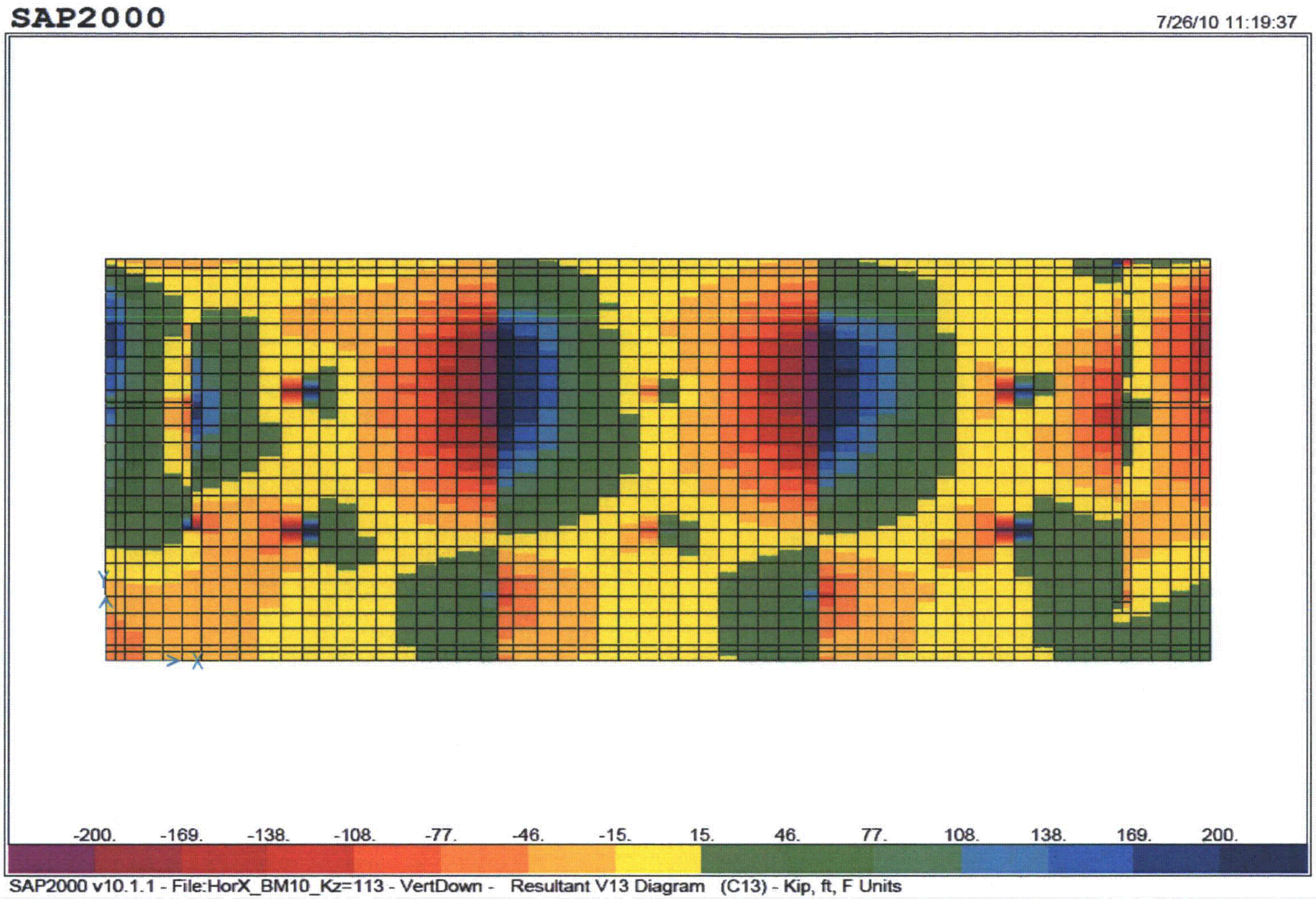


Figure 03.07.01-28.24: Resultant Out-of-Plane Shear V13 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

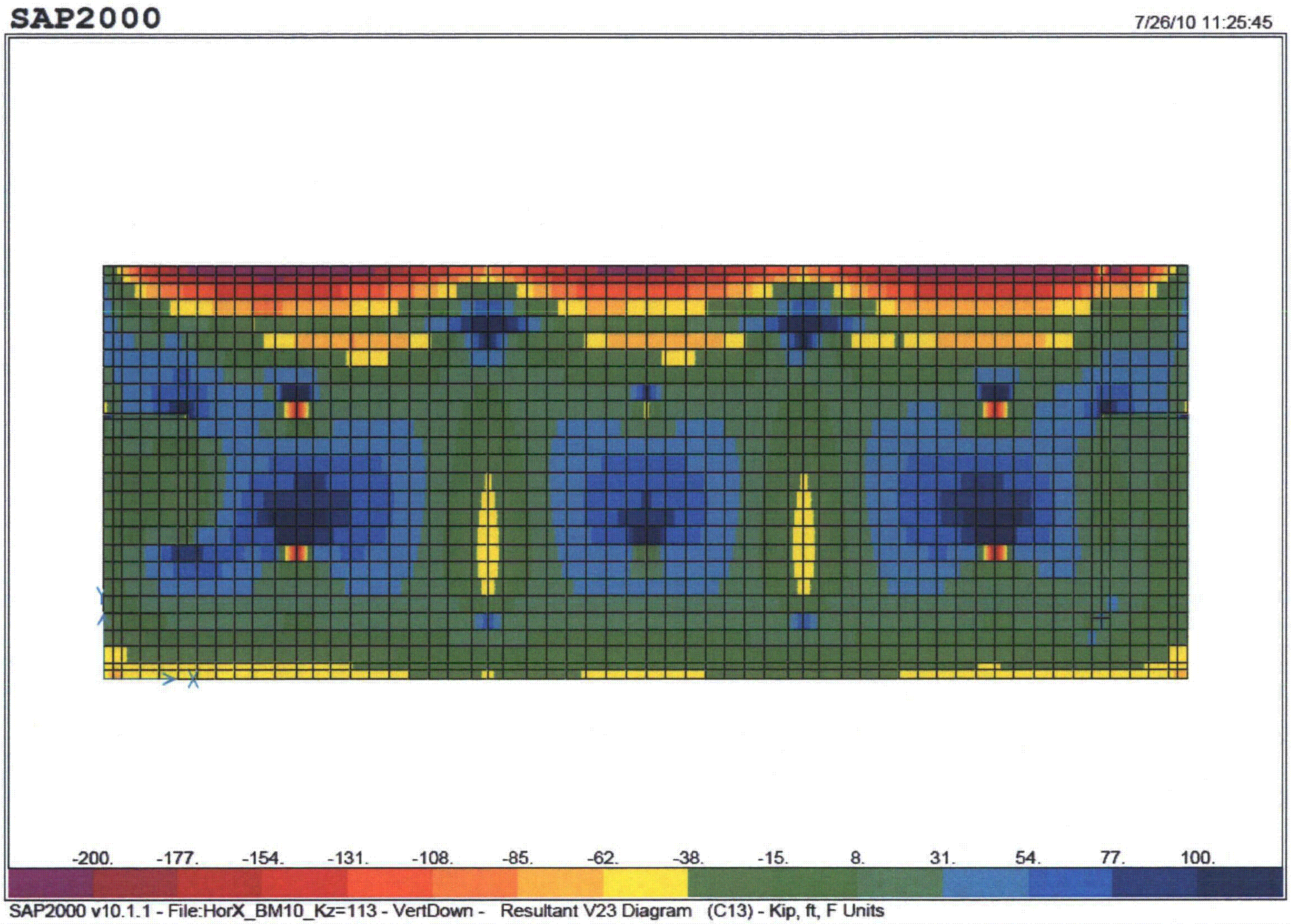


Figure 03.07.01-28.25: Resultant Out-of-Plane Shear V23 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Down, and Moment M_x

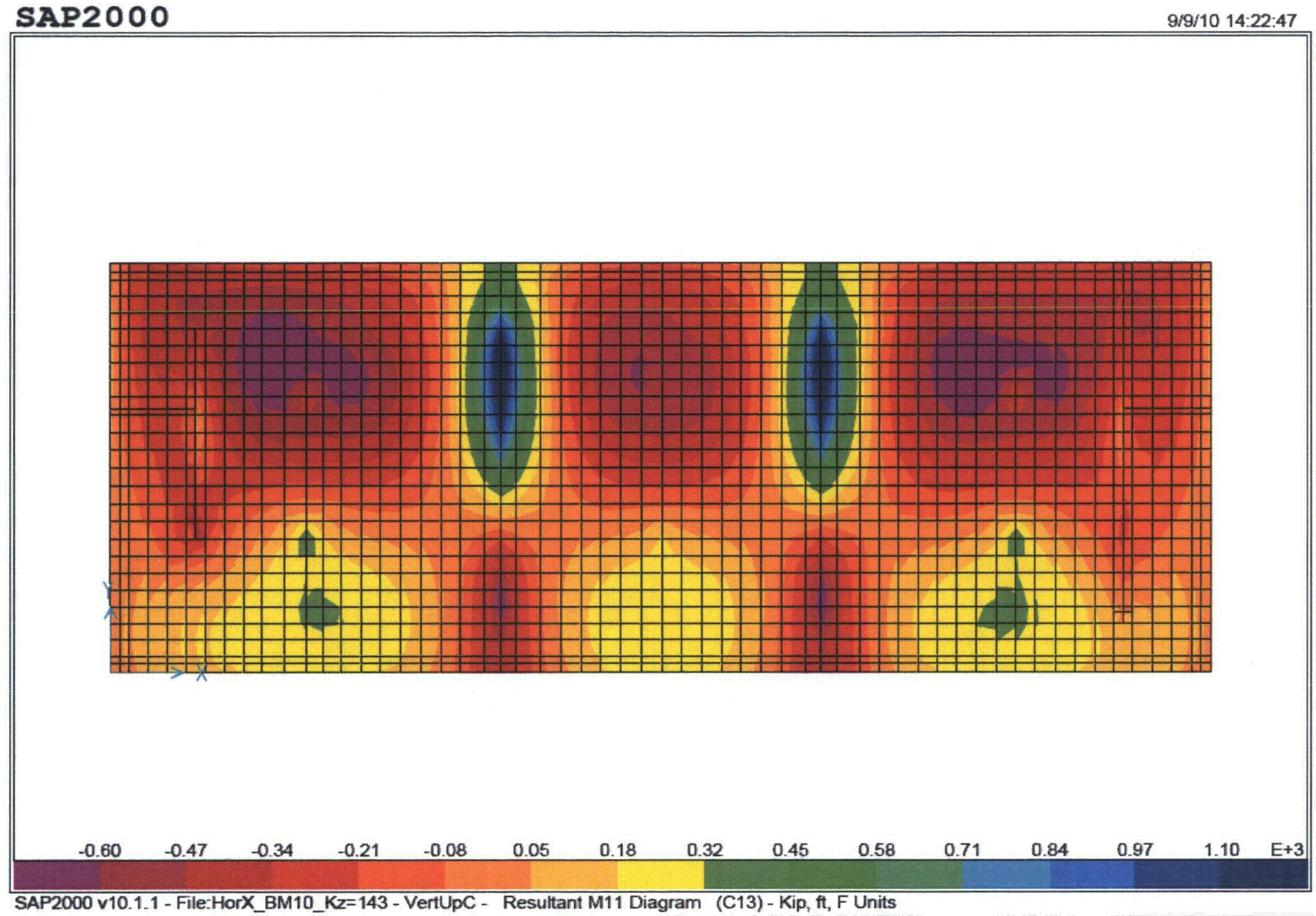


Figure 03.07.01-28.26: Resultant Out-of-Plane Moment M11 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_x

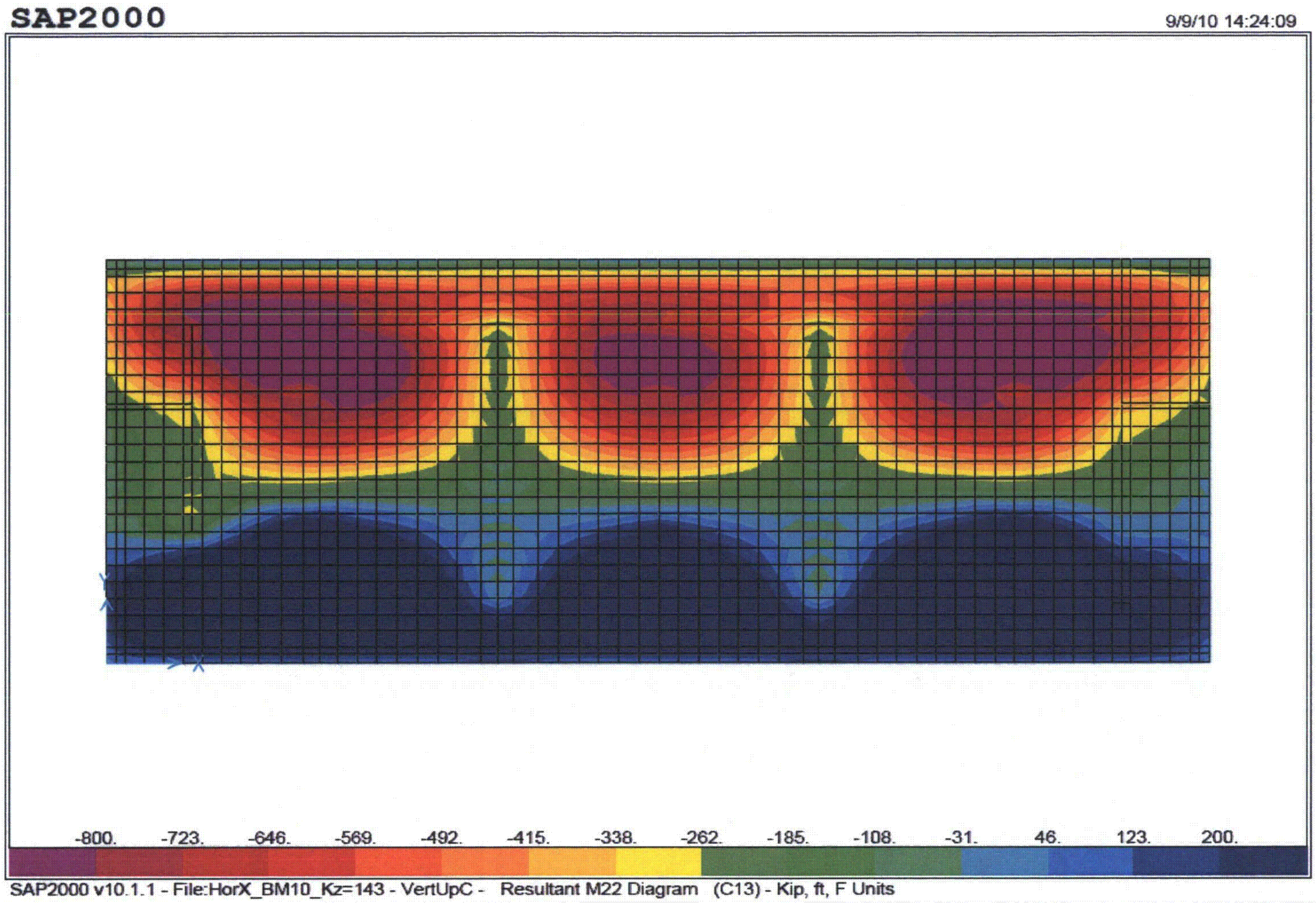


Figure 03.07.01-28.27: Resultant Out-of-Plane Moment M22 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_x

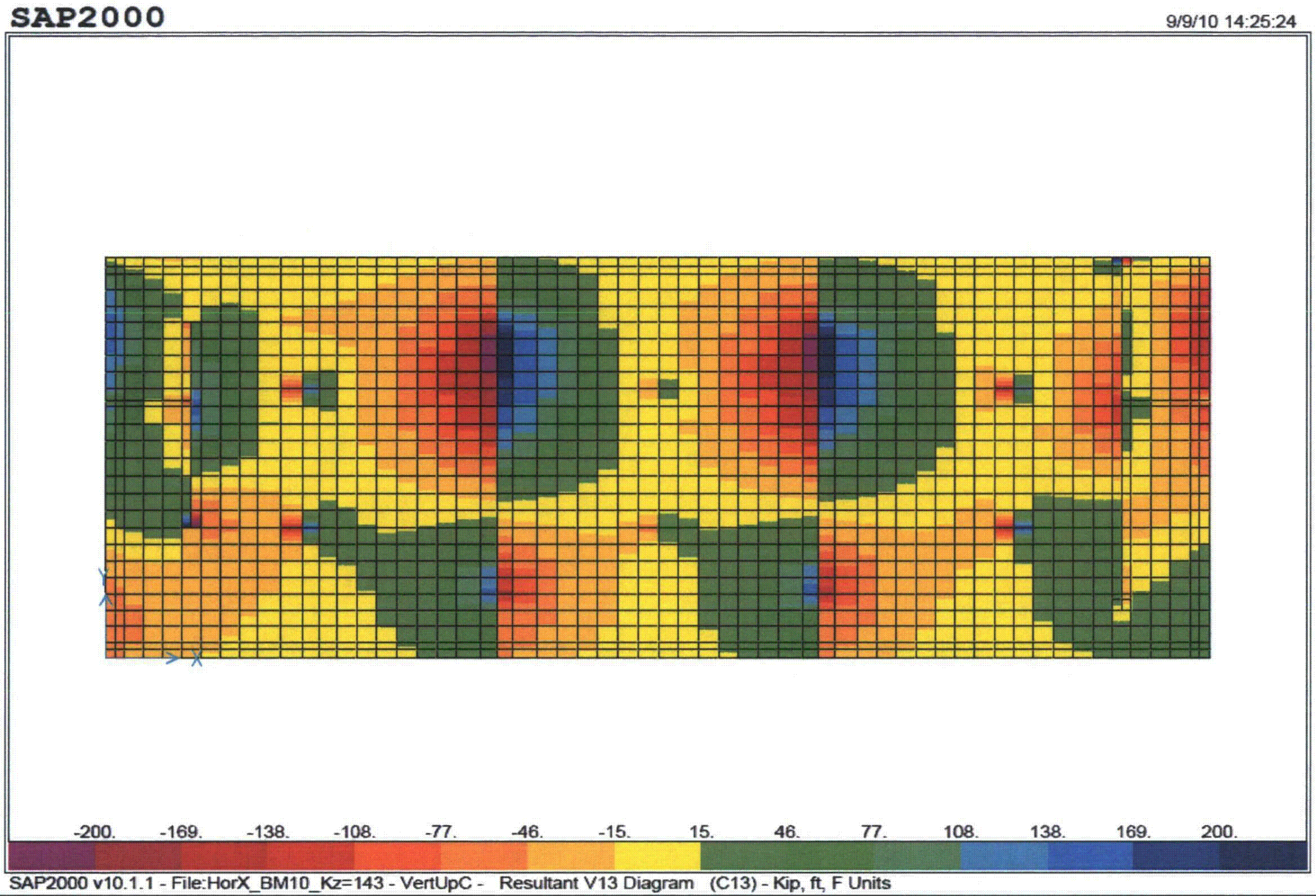


Figure 03.07.01-28.28: Resultant Out-of-Plane Shear V13 Diagram
(Using DCD Spring Constants)
Dead Load, Vertical Excitation Up, and Moment M_x

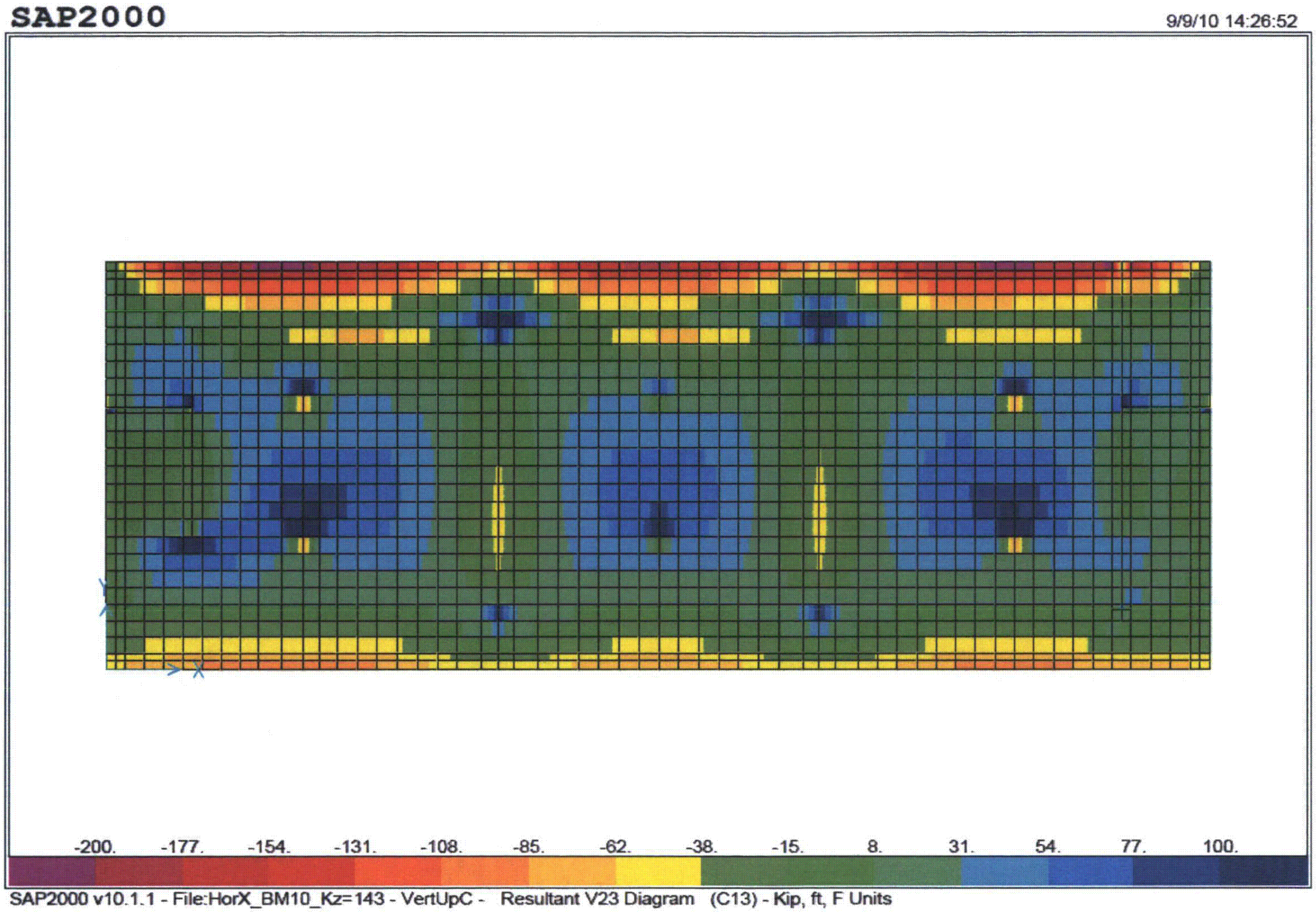


Figure 03.07.01-28.29: Resultant Out-of-Plane Shear V23 Diagram (Using DCD Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_x

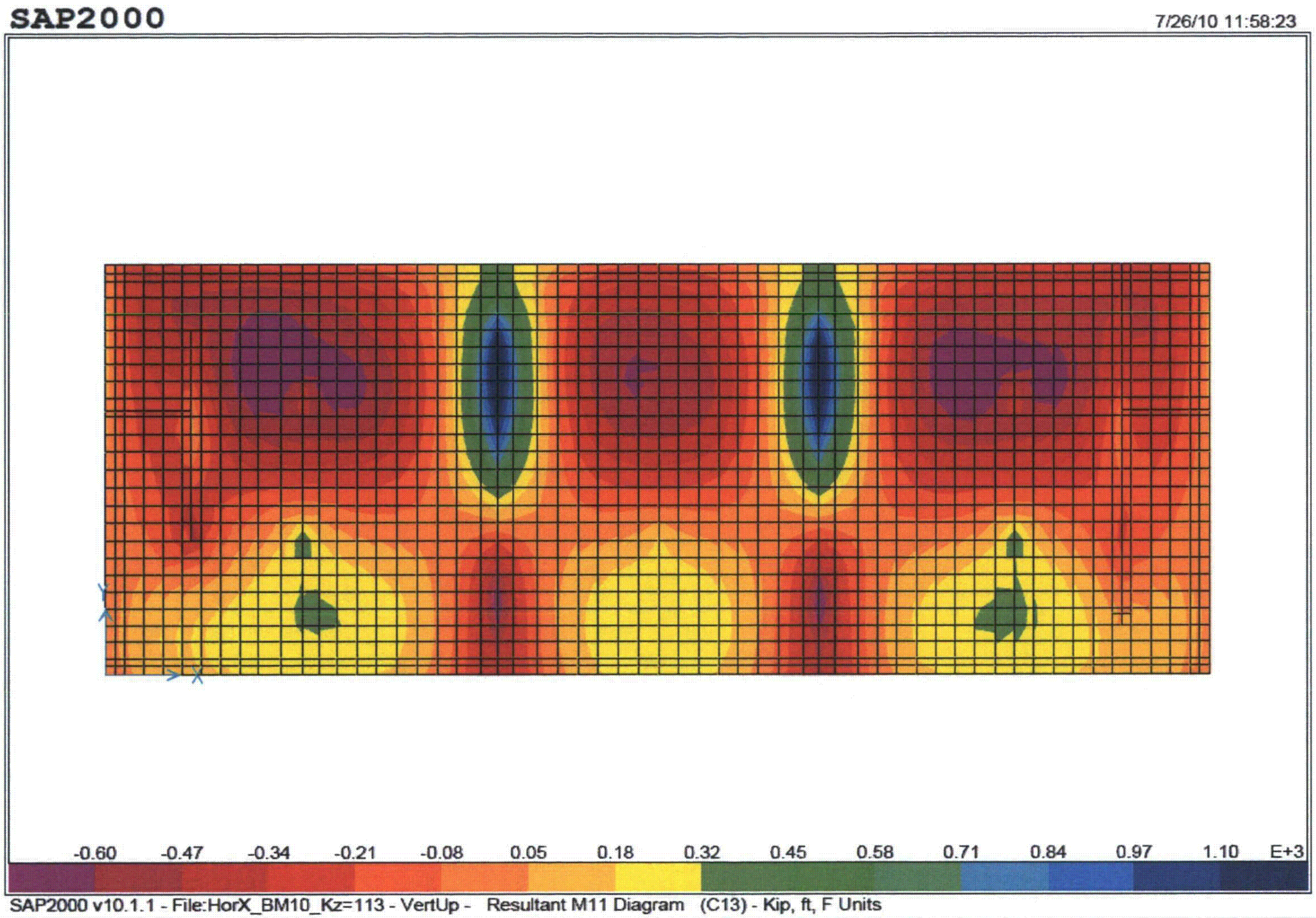


Figure 03.07.01-28.30: Resultant Out-of-Plane Moment M11 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_x

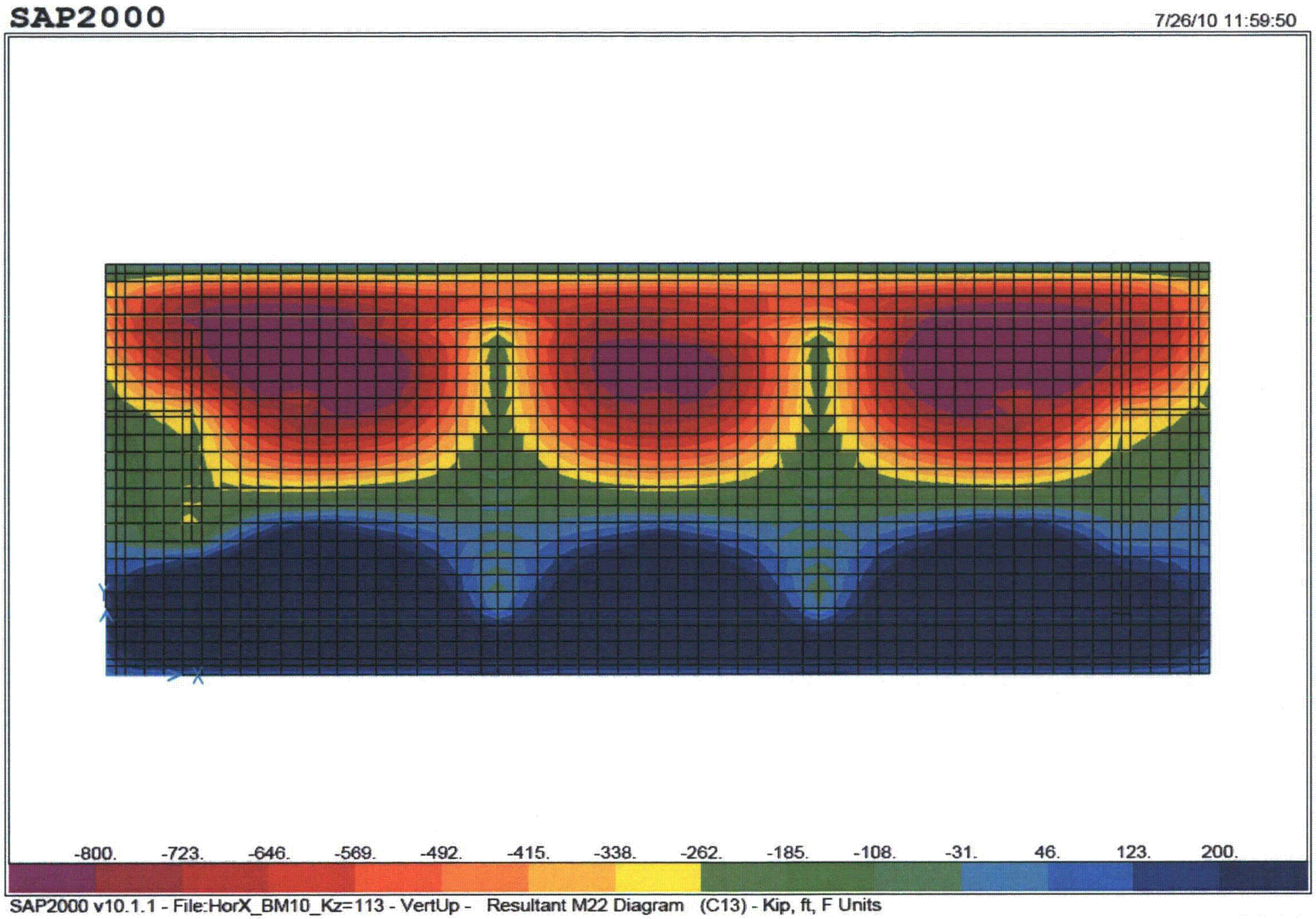


Figure 03.07.01-28.31: Resultant Out-of-Plane Moment M22 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_x

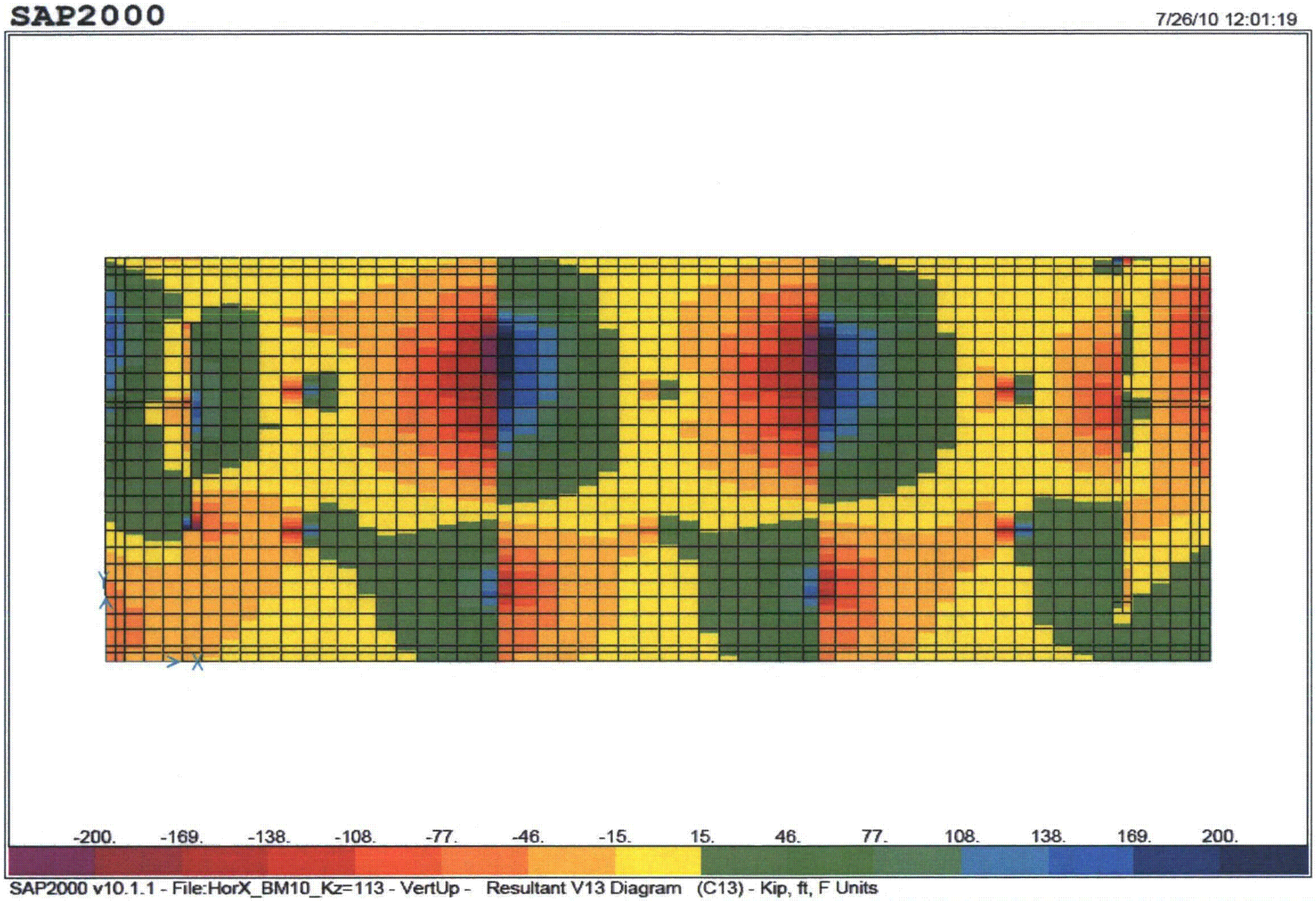


Figure 03.07.01-28.32: Resultant Out-of-Plane Shear V13 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_x

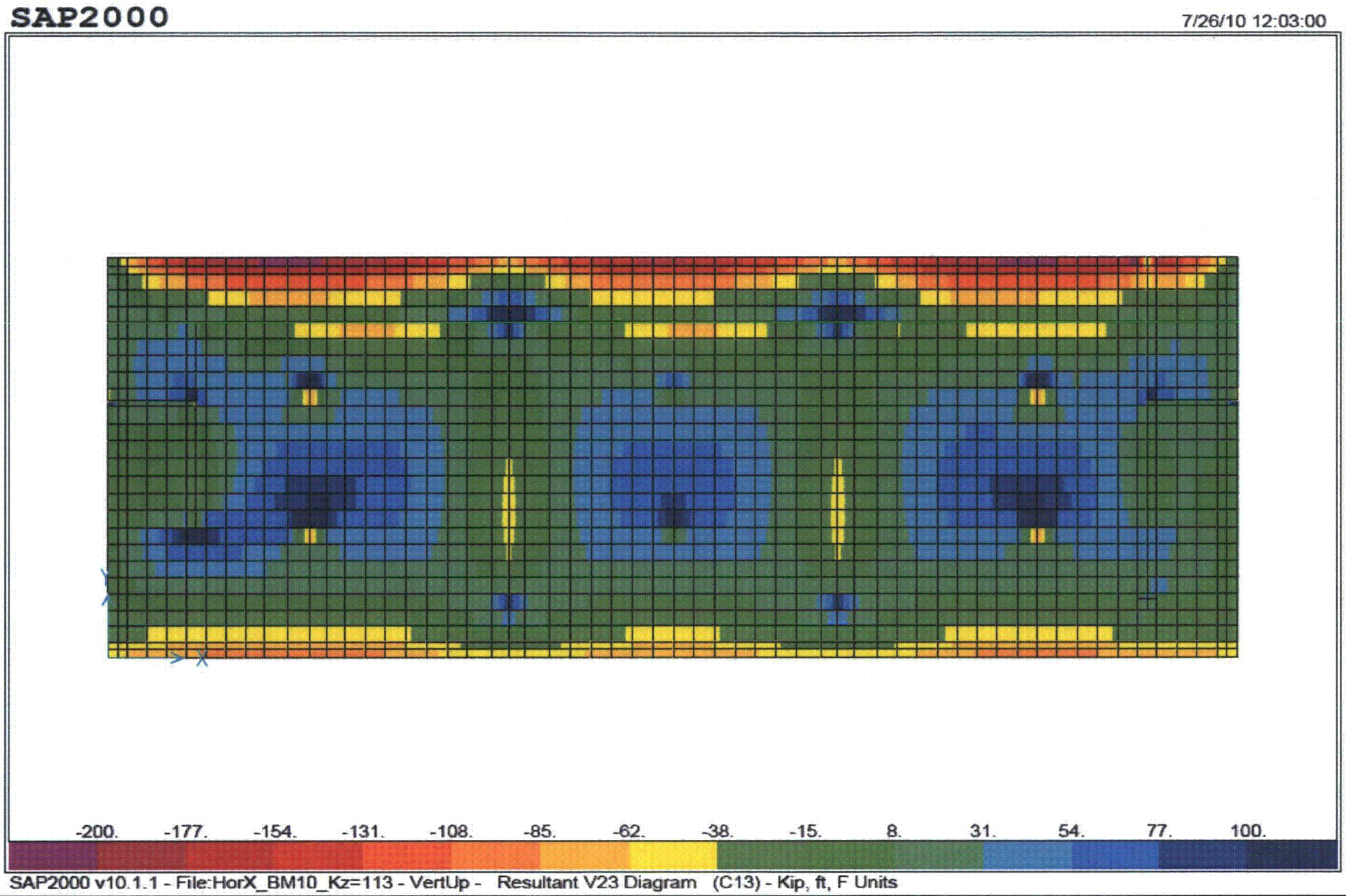


Figure 03.07.01-28.33: Resultant Out-of-Plane Shear V23 Diagram (Using Lower Range Site-Specific Spring Constants) Dead Load, Vertical Excitation Up, and Moment M_x

RAI 03.07.02-23

QUESTION:

Follow-up Question to RAI 03.07.02-14 (STP-NRC-100036)

10CFR50, Appendix S requires that evaluation for SSE must take into account soil-structure interaction (SSI) effects. In the response to Item 6 of RAI 03.07.02-14, the applicant has stated that for evaluating the effect of soil separation from the walls, the method recommended in Section 3.3.1.9 of ASCE 4-98 was used. The ASCE 4-98 criteria is a general guidance, and NRC has not endorsed this guidance for estimating the depth of soil separation for Seismic Category I structures, such as UHS Basin and RSW Pump House. As such, the applicant is requested to provide additional basis to justify that use of ASCE guidance is conservative in estimating the depth of soil separation. In providing the justification, the applicant may obtain the dynamic soil pressures calculated along the height of each soil-bearing wall from the SSI analysis of the UHS Basin and RSW Pump House, and compare the results with the static soil pressures acting on the walls. From this comparison, the applicant may calculate the net negative pressure exerted on each wall, and use the results to estimate the depth of soil separation from the walls and compare it with that obtained from ASCE guidance to demonstrate acceptability. The staff needs this justification to ensure proper consideration of effect of potential soil separation in SSI evaluation.

RESPONSE:

In the Soil-Structure Interaction (SSI) analyses of the Ultimate Heat Sink (UHS) basin and Reactor Service Water (RSW) Pump House, based on the guidelines provided in Section 3.3.1.9 of ASCE 4-98 a depth of 20 ft was considered for the soil separation case. In this analysis, the soil was disconnected from the walls to a depth of 20 ft to elevation 14 ft MSL on all sides of the structure. The depth of 20 ft aligns with the top of the UHS basin basemat and was considered to be a bounding case, providing more separation than is expected.

As suggested in the RAI, to justify the 20 ft depth of soil separation, the dynamic soil pressures calculated from SSI analyses in SASSI2000 are compared with the static soil pressures. Where the dynamic soil pressure is less than the static soil pressure, the soil will remain in contact and where the dynamic soil pressure exceeds the static soil pressure, the soil may separate from the structure.

The dynamic SSI soil pressure is calculated from the spring elements connecting the structure to the soil in the SSI model. For each analysis the peak spring force over all time steps is calculated in SASSI2000 for each direction of input motion. The results from the three input motions are combined by Square Root of the Sum of the Squares (SRSS) method. The soil pressure at each elevation is calculated as the sum of peak force in the springs at each elevation divided by the total representative tributary area. This pressure is enveloped over all soil cases.

The static soil pressure is calculated using an at-rest pressure coefficient, $K_o = 1 - \sin(\phi)$, where ϕ is the soil internal friction angle. The static soil pressure is calculated as the pressure coefficient multiplied by the soil unit weight and the depth. At rest pressure represents soil pressure where the wall is rigid and unmoving. The internal friction angle for the backfill is expected to be between 30 and 40 degrees with the most likely internal friction angle being 36 degrees. For internal friction angles of 30°, 36° and 40° the at- rest pressure coefficients are 0.5, 0.412 and 0.357 respectively.

Static soil pressure could be calculated based on the average of active pressure on one side and a portion of passive pressure on the other side. Active pressure occurs where the structure yields or moves away from the soil, and passive pressure occurs when the structure moves into the soil with the maximum value achieved when a soil failure state is achieved. The average is used because soil separation is modeled on both sides while separation would occur only on one side at a given time. Calculation of the amount of passive pressure that can be developed is not straightforward. Alternately, knowing that soil separation will occur on one side but not the other, a value of twice the active pressure could be used for the estimation of soil separation height. Active pressure coefficient, $K_a = \tan^2(45° - \phi/2)$. For internal friction angles of 30° and 40° twice the active pressure coefficients are 0.667 and 0.435 respectively. Since the at-rest pressure is always less than twice the active pressure, the at-rest pressure is conservatively used for determination of maximum soil separation.

Static soil pressure is calculated for two conditions:

- Groundwater table below the Pump House basemat, using a moist soil unit weight of 120 pcf,
- Groundwater table 6 ft below grade (maximum groundwater table), using a moist soil unit weight of 120 pcf, and a saturated unit weight conservatively equal to the moist soil unit weight. Soil pressure using the effective soil unit weight (buoyancy effect due to the groundwater) and the hydrostatic pressure.

Figures 03.07.02-23.1 through 03.07.02-23.7 show the comparison of at-rest soil pressure to SSI soil pressure with the groundwater table being below the Pump House basemat. Figures 03.07.02-23.8 through 03.07.02-23.14 show the comparison of at-rest soil pressure to SSI soil pressure with the groundwater table at 6 ft below grade. Tables 03.07.02-23.1 and 03.07.02-23.2 show the depth of separation calculated for the internal soil friction angles of 30°, 36°, and 40° for the cases with groundwater table below the Pump House basemat and for the groundwater table six feet below grade respectively. The results show that the 20 ft separation depth used for the SSI analysis case with separated soil is justified.

No additional COLA revision is required as a result of this response.

Table 03.07.02-23.1: Depth of Separation Calculated by Intersection of At-Rest Soil Pressure and Enveloped Mean SSI Soil Pressures

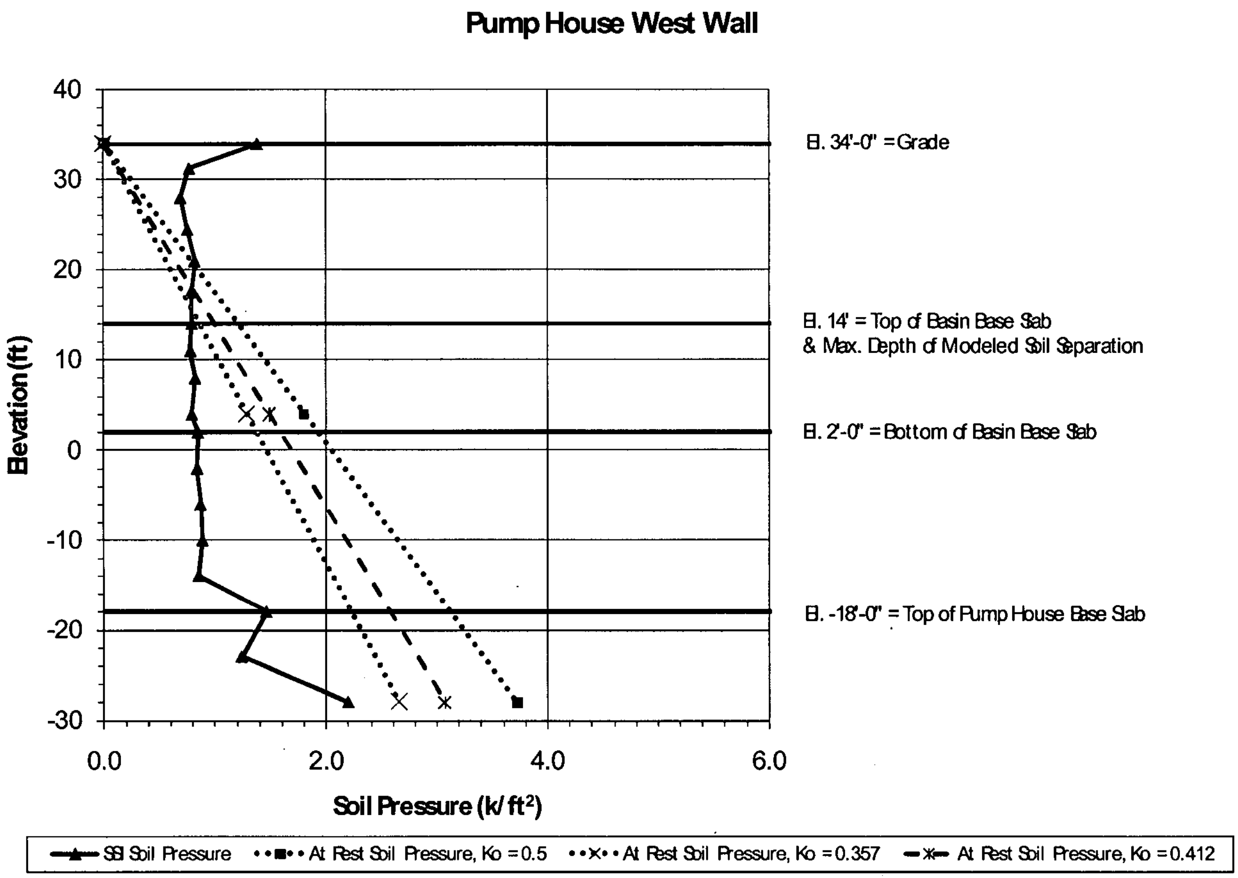
(Groundwater Table below Pump House Basemat)

	Ko = 0.5		Ko = 0.412		Ko = 0.357	
	Separation Depth (ft)	Separation Elevation (ft)	Separation Depth (ft)	Separation Elevation (ft)	Separation Depth (ft)	Separation Elevation (ft)
Pump House West Wall	12.0	22.0	16.0	18.0	18.0	16.0
Basin West Wall	16.0	18.0	17.5	16.5	19.0	15.0
Pump House East Wall	13.5	20.5	16.0	18.0	19.0	15.0
Basin East Wall	16.5	17.5	18.0	16.0	19.7	14.3
Basin North Wall	14.0	20.0	15.0	19.0	16.0	18.0
Pump House North Wall	10.0	24.0	13.0	21.0	15.0	19.0
Basin South Wall	14.0	20.0	14.5	19.5	15.5	18.5

Table 03.07.02-23.2: Depth of Separation Calculated by Intersection of At-Rest Soil Pressure and Enveloped Mean SSI Soil Pressures

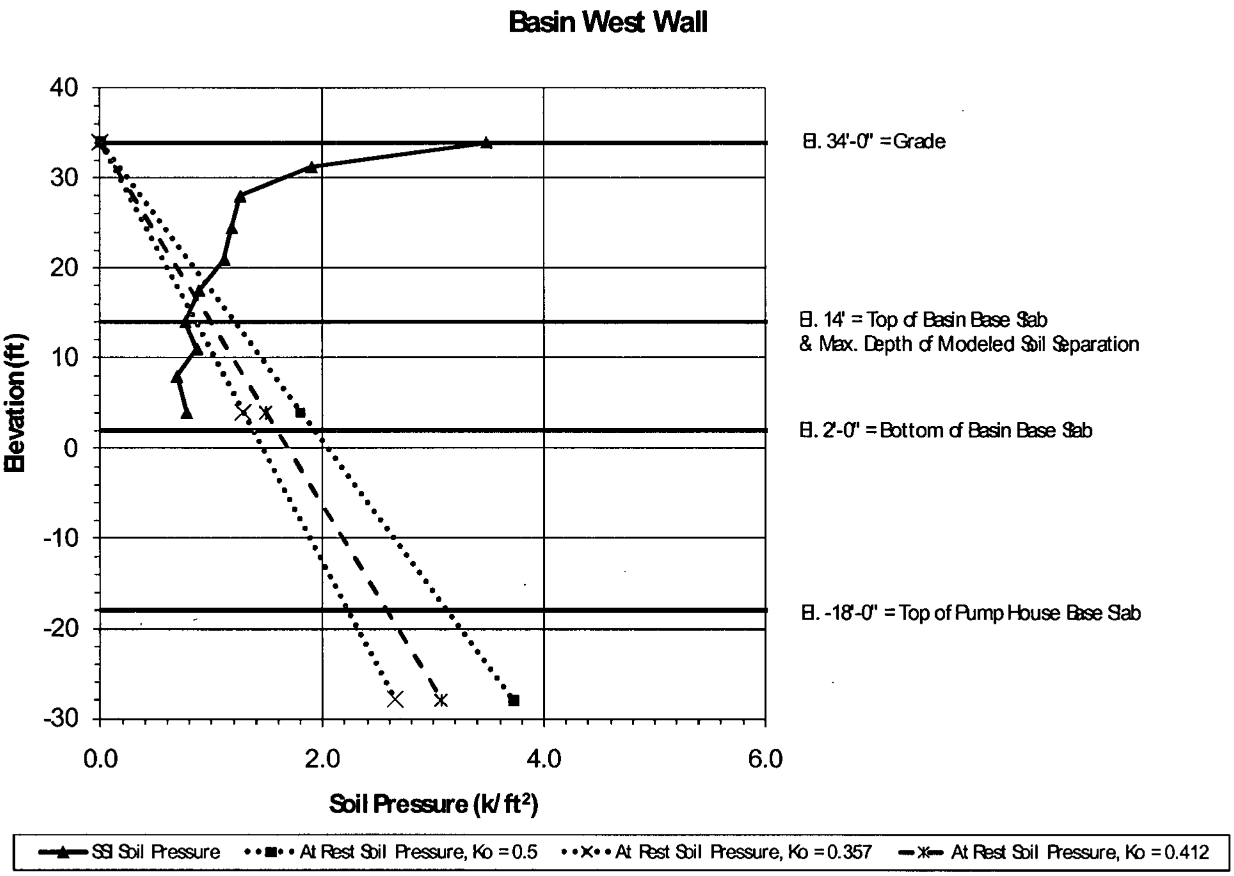
(Groundwater Table Six Feet below Grade)

	Ko = 0.5		Ko = 0.412		Ko = 0.357	
	Separation Depth (ft)	Separation Elevation (ft)	Separation Depth (ft)	Separation Elevation (ft)	Separation Depth (ft)	Separation Elevation (ft)
Pump House West Wall	10.5	23.5	11.8	22.2	12.7	21.3
Basin West Wall	13.7	20.3	14.4	19.6	14.9	19.1
Pump House East Wall	10.0	24.0	11.4	22.6	11.4	22.6
Basin East Wall	14.4	19.6	15.0	19.0	15.4	18.6
Basin North Wall	11.5	22.5	12.6	21.4	13.2	20.8
Pump House North Wall	8.2	25.8	9.2	24.8	10.0	24.0
Basin South Wall	11.0	23.0	12.0	22.0	12.7	21.3



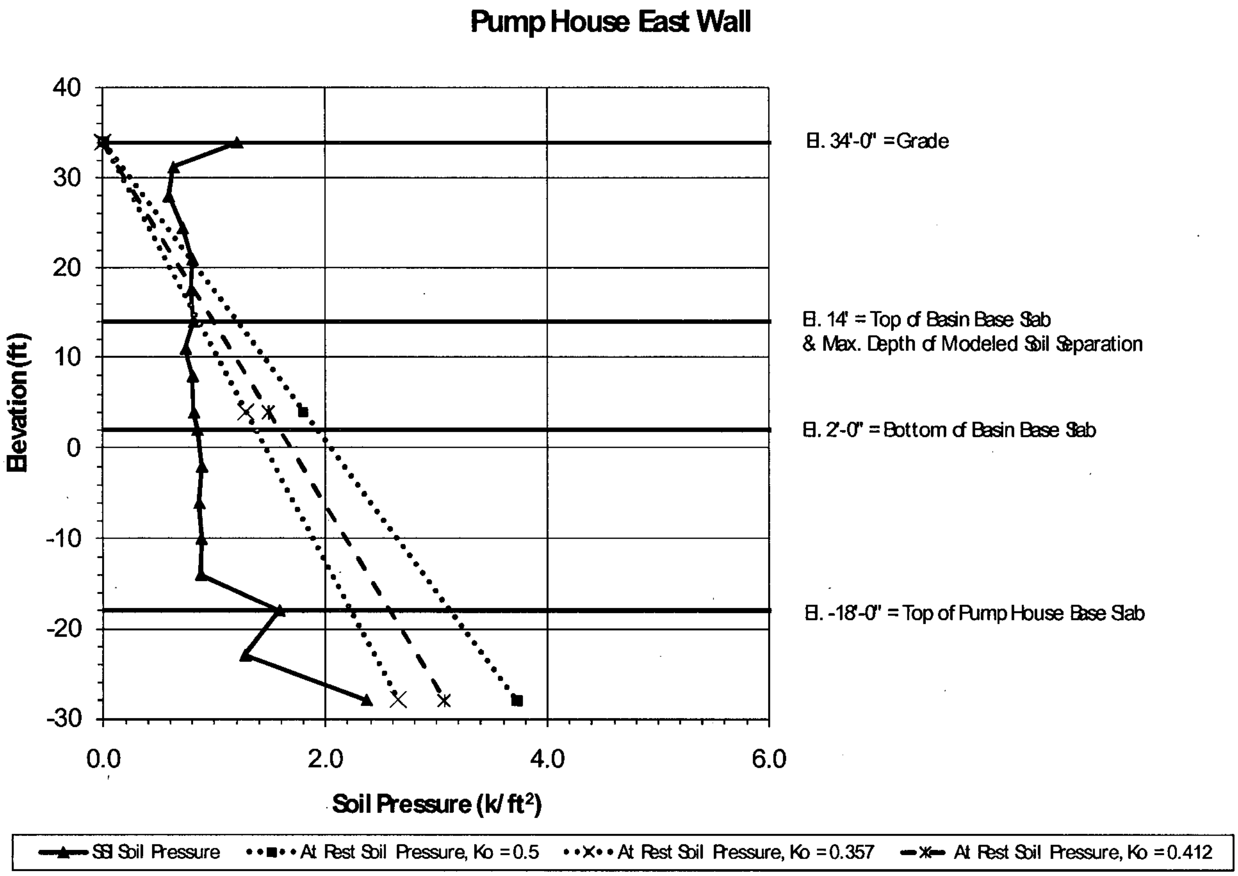
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.1: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table below Pump House Basemat) Pump House West Wall



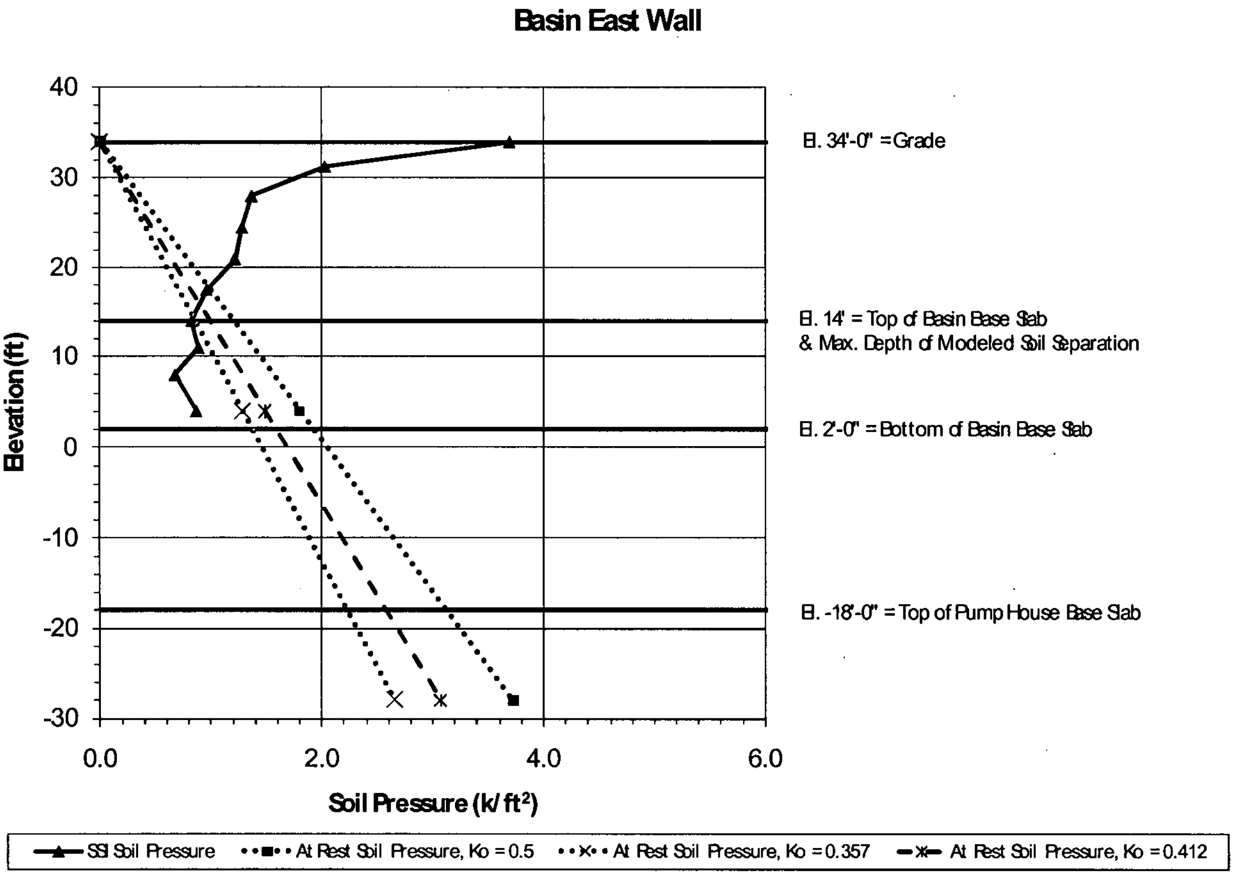
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.2: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table below Pump House Basemat) UHS Basin West Wall



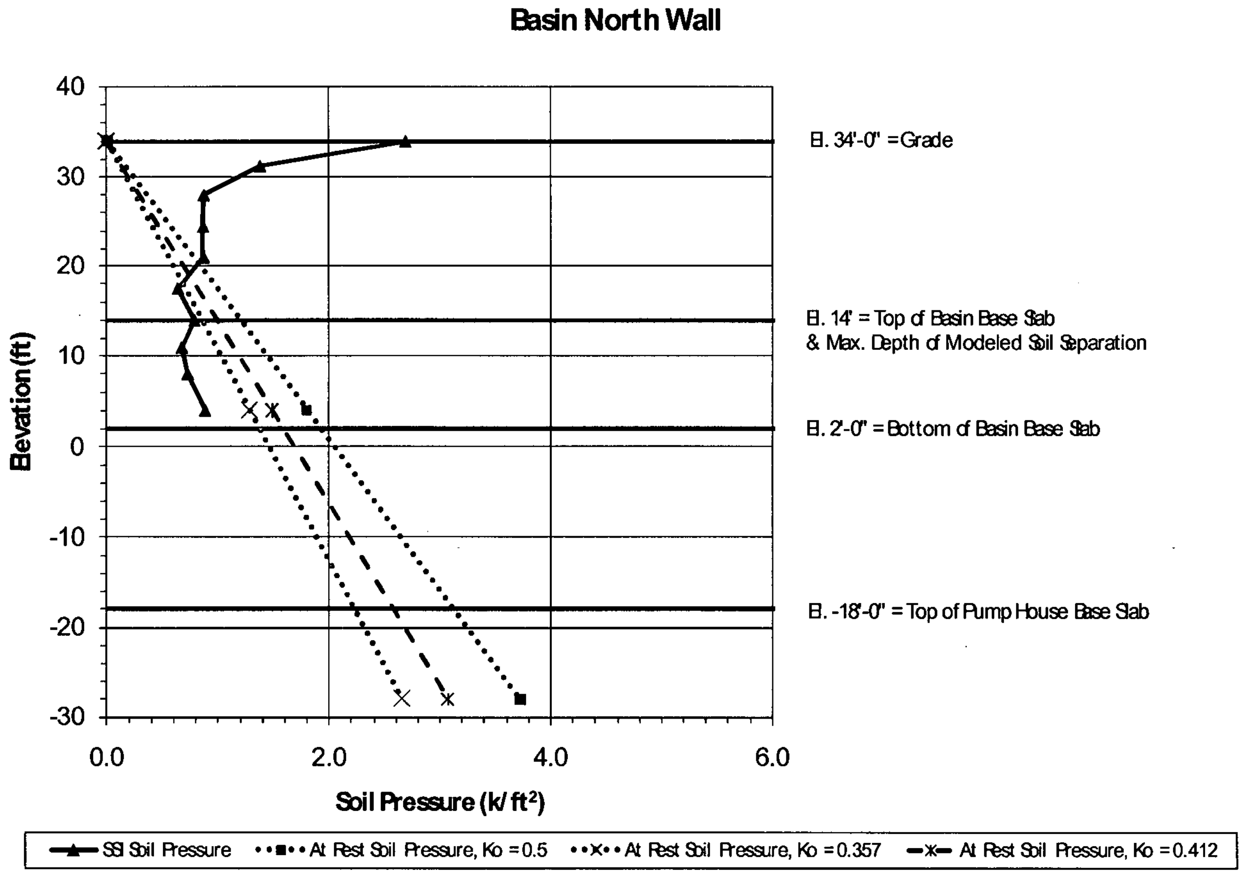
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.3: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table below Pump House Basemat) Pump House East Wall



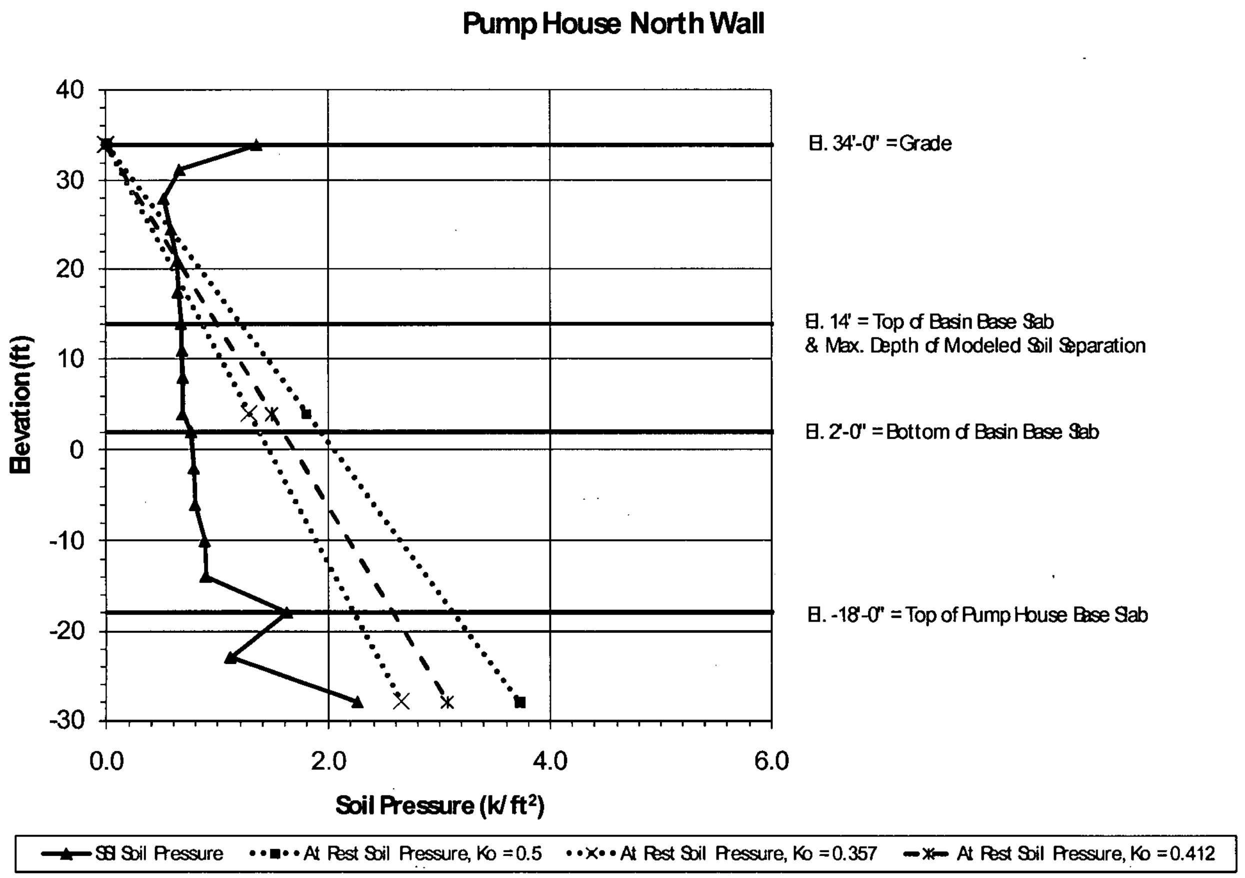
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.4: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table below Pump House Basemat) UHS Basin East Wall



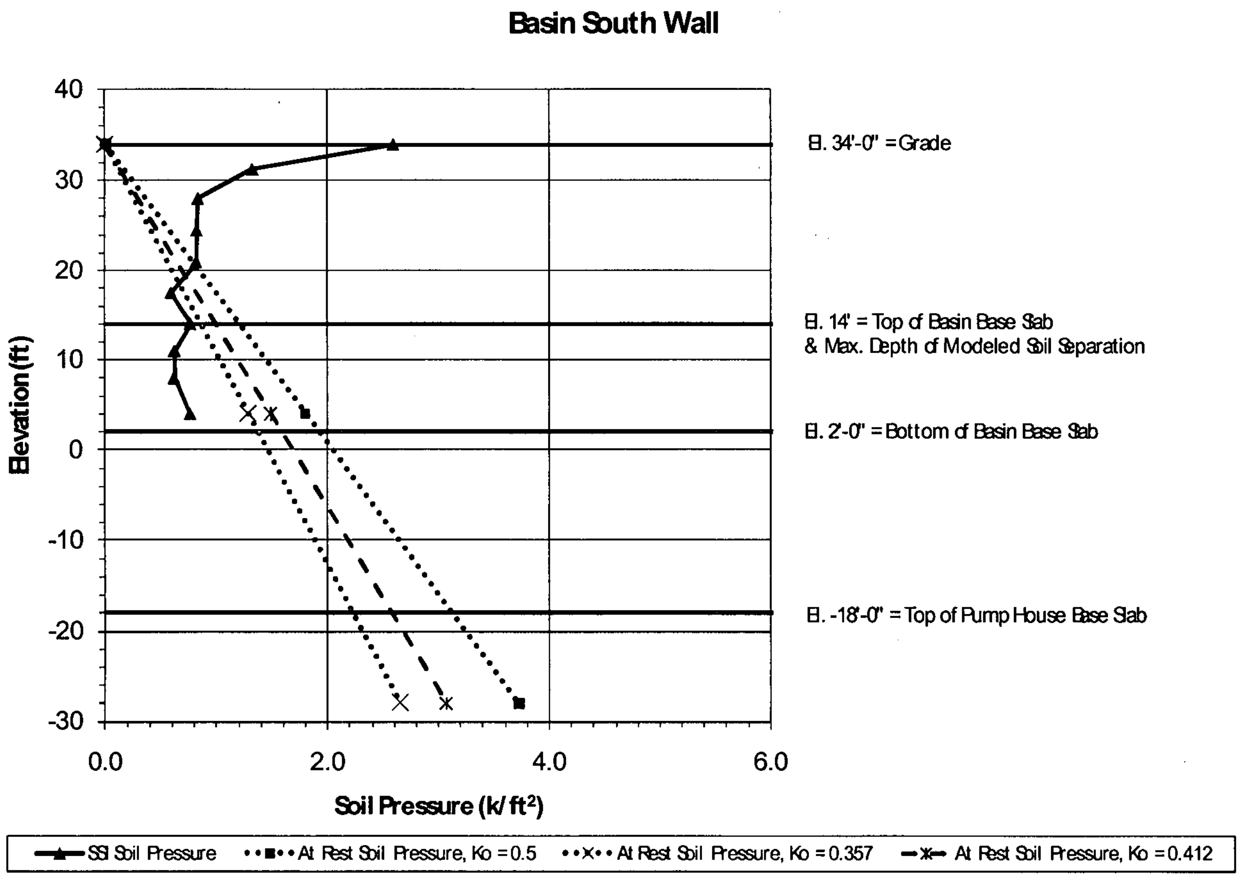
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.5: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table below Pump House Basemat) UHS Basin North Wall



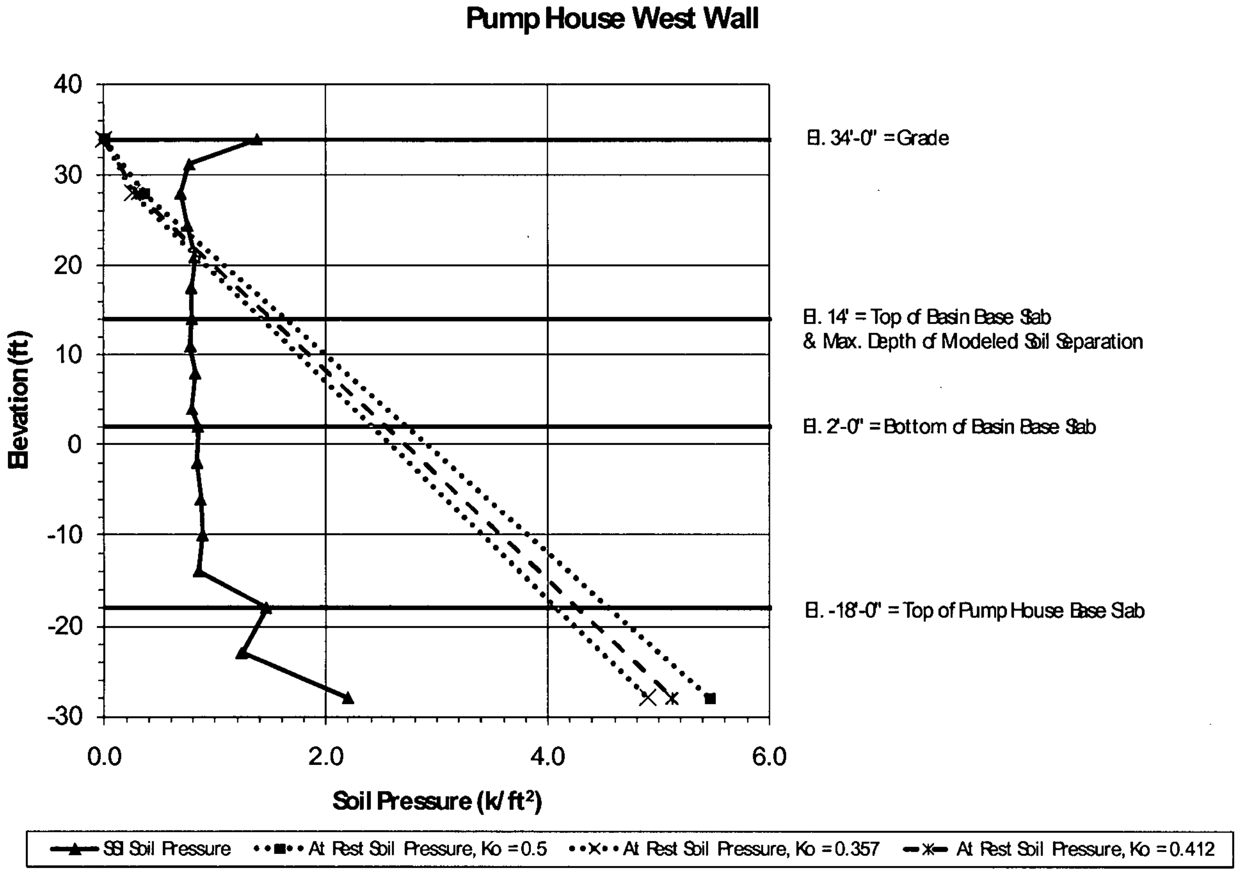
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.6: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table below Pump House Basemat) Pump House North Wall



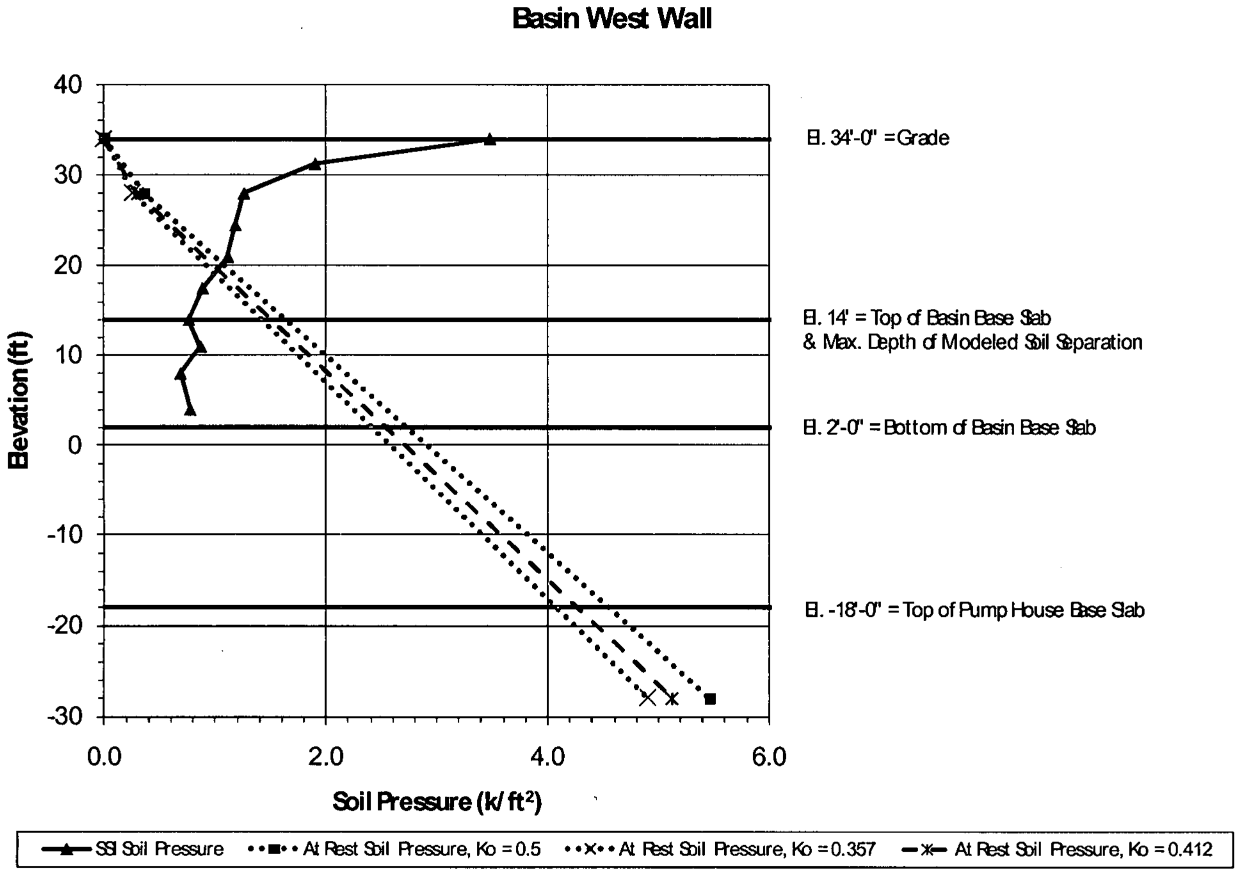
Note: Ko = 0.5 corresponds with $\phi = 30^{\circ}$
Ko = 0.412 corresponds with $\phi = 36^{\circ}$
Ko = 0.357 corresponds with $\phi = 40^{\circ}$

Figure 03.07.02-23.7: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table below Pump House Basemat) UHS Basin South Wall



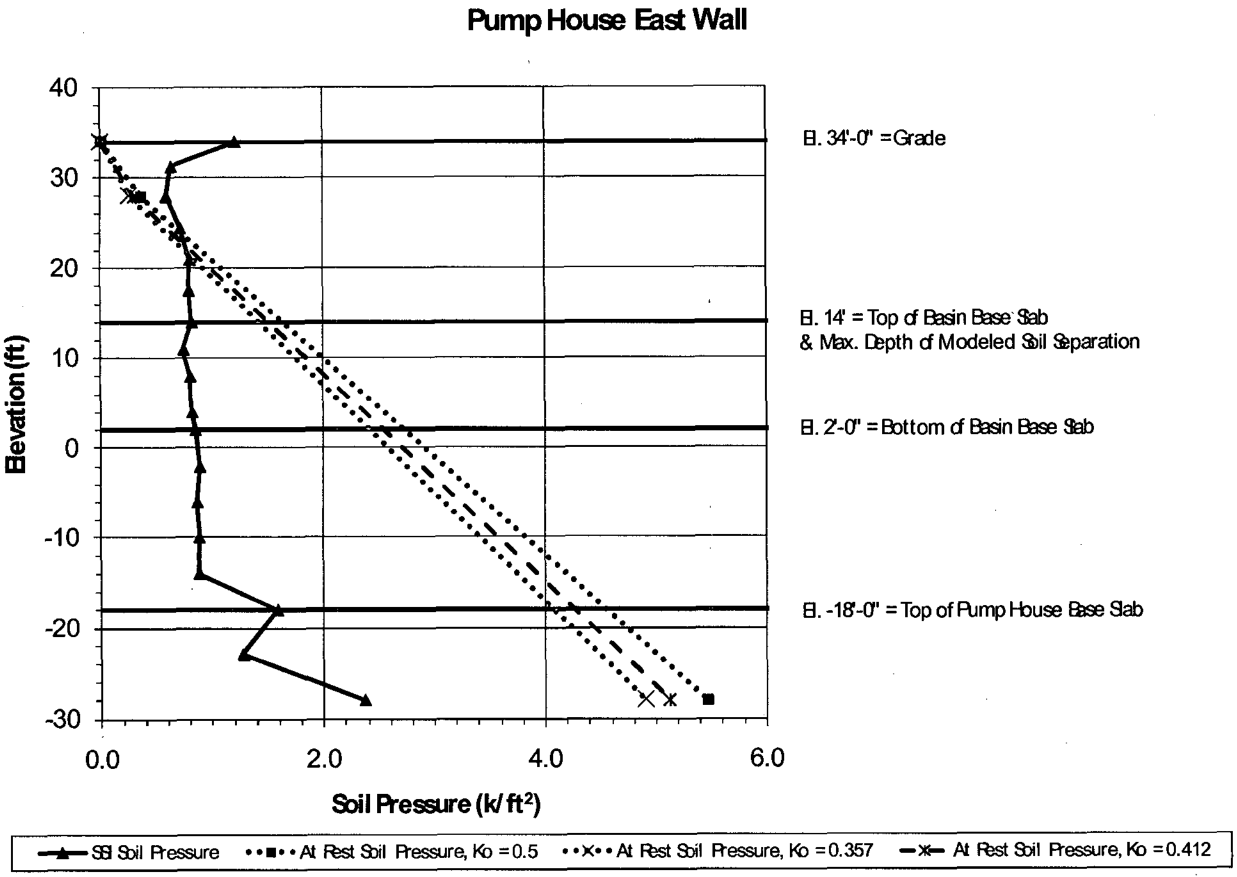
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.8: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table Six Feet below Grade) Pump House West Wall



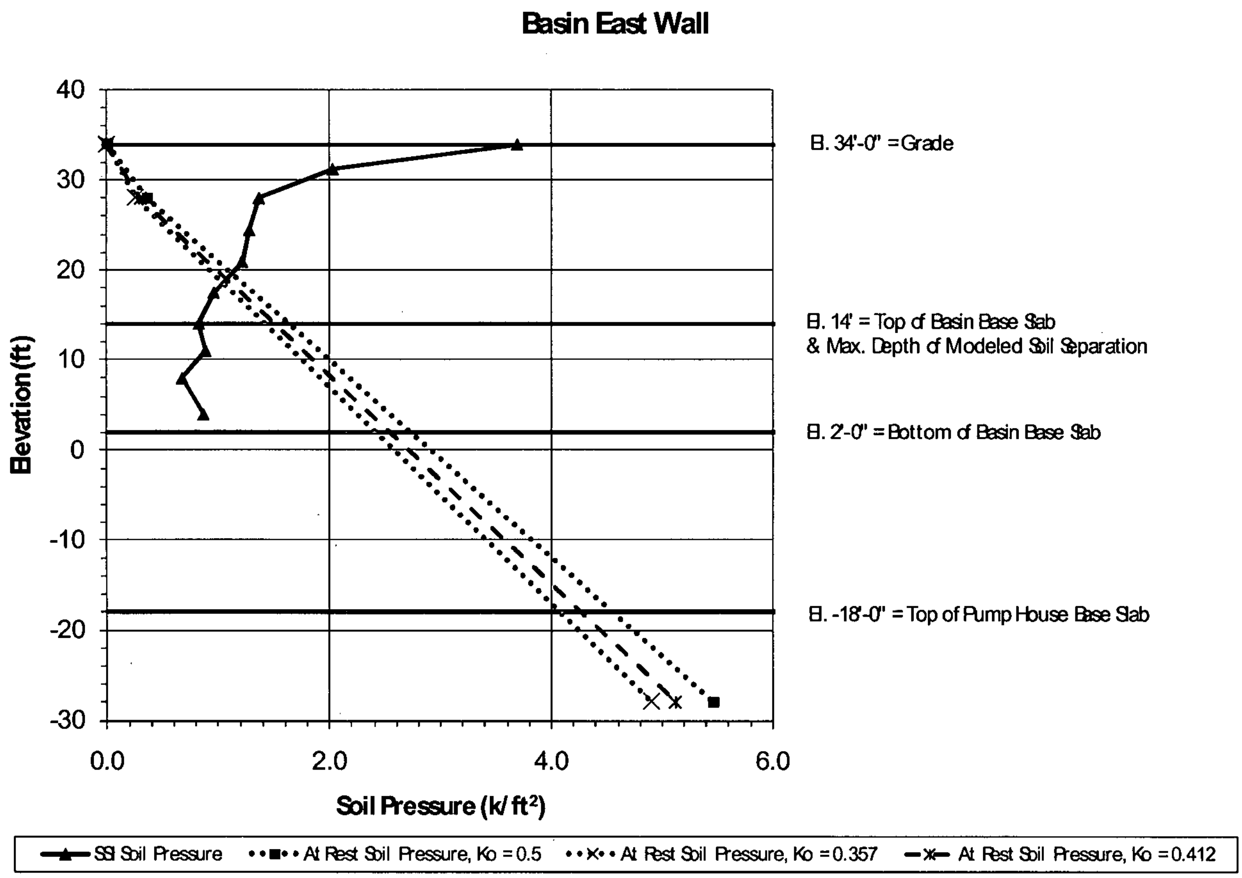
Note: Ko = 0.5 corresponds with $\phi = 30^{\circ}$
Ko = 0.412 corresponds with $\phi = 36^{\circ}$
Ko = 0.357 corresponds with $\phi = 40^{\circ}$

Figure 03.07.02-23.9: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table Six Feet below Grade) UHS Basin West Wall



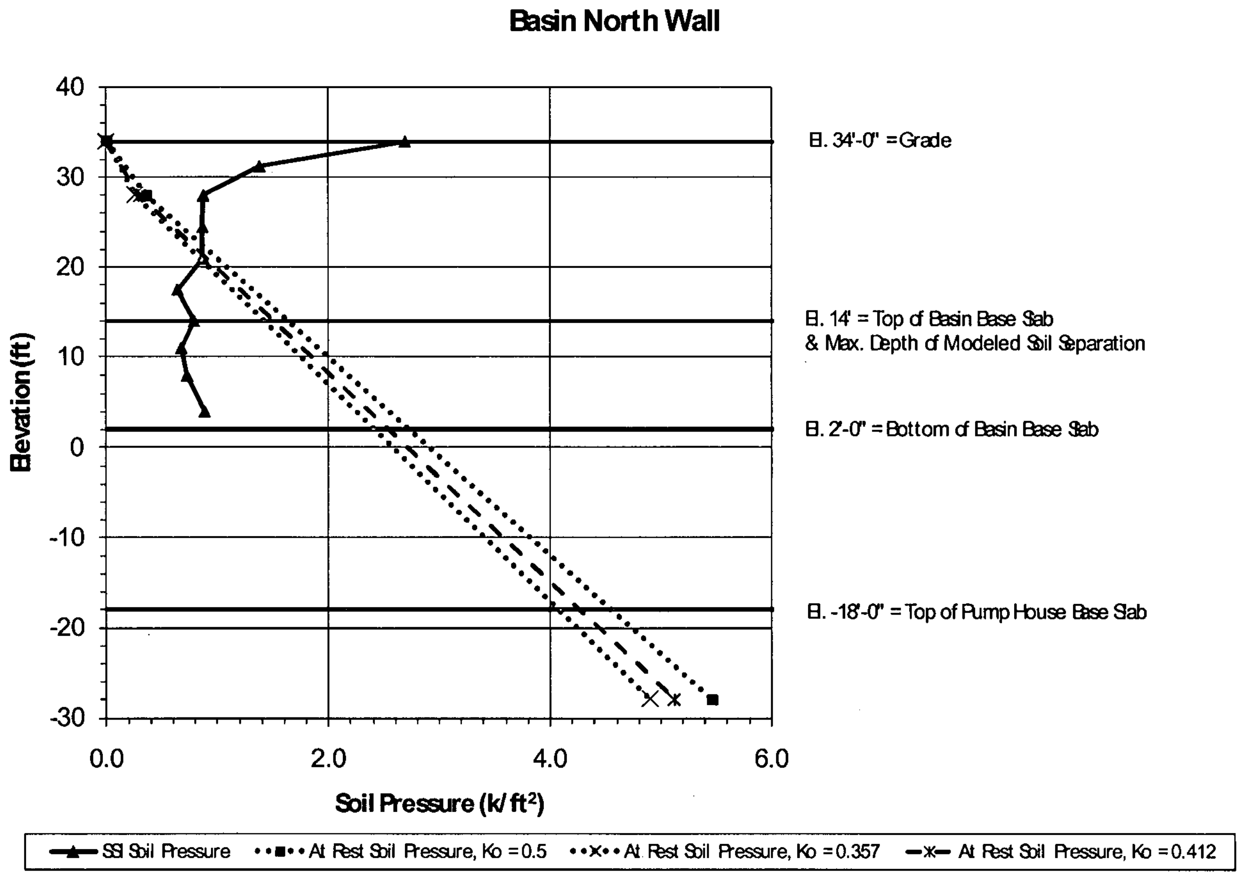
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.10: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table Six Feet below Grade) Pump House East Wall



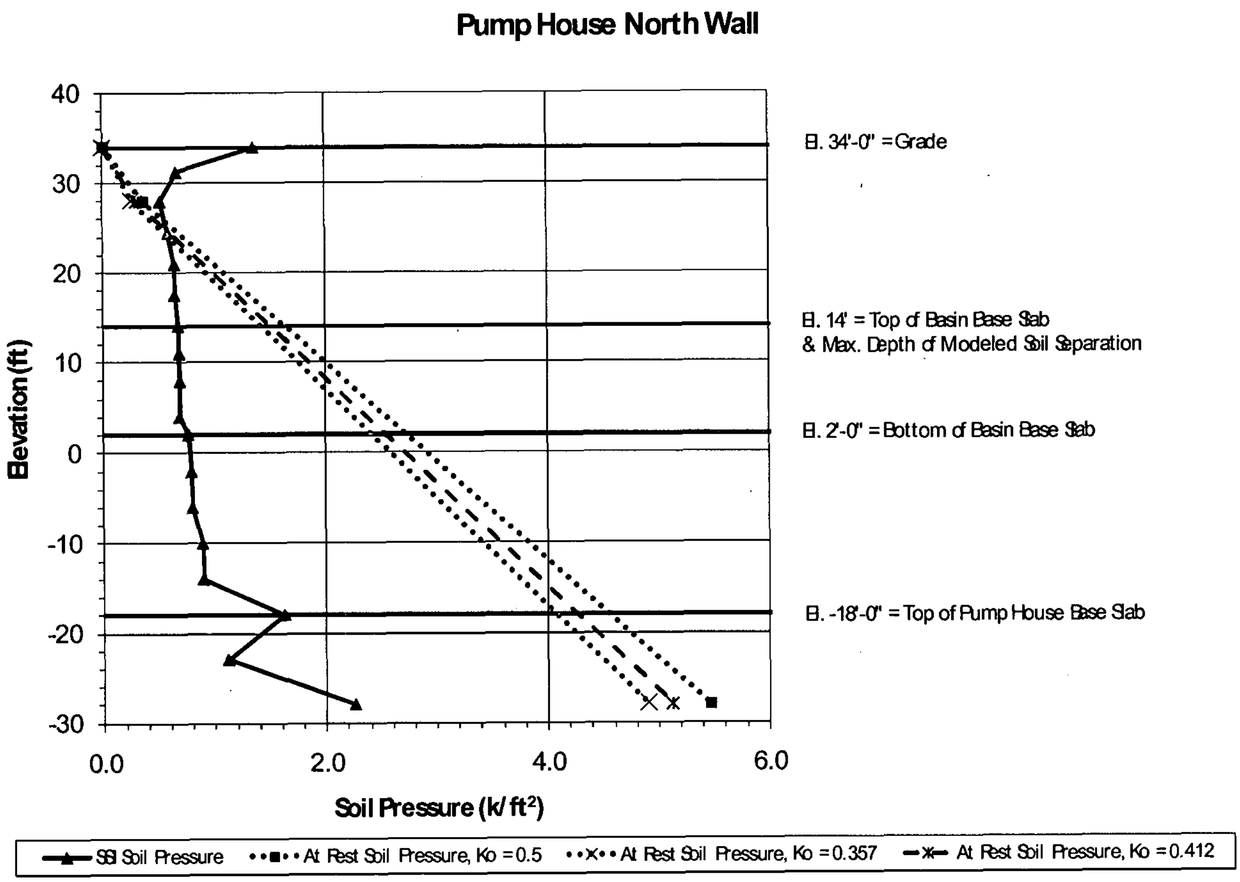
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.11: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table Six Feet below Grade) UHS Basin East Wall



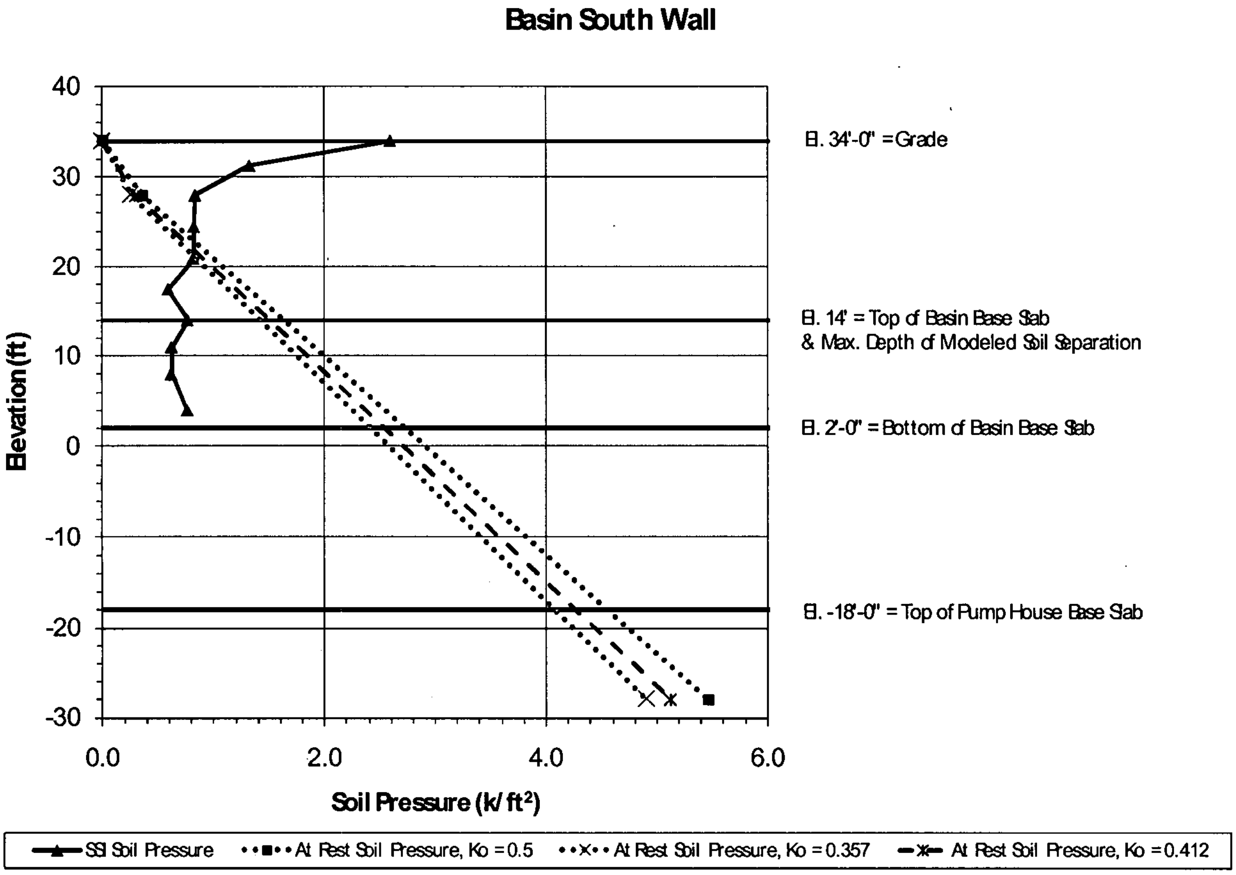
Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.12: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table Six Feet below Grade) UHS Basin North Wall



Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.13: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table Six Feet below Grade) Pump House North Wall



Note: Ko = 0.5 corresponds with $\phi = 30^{o}$
Ko = 0.412 corresponds with $\phi = 36^{o}$
Ko = 0.357 corresponds with $\phi = 40^{o}$

Figure 03.07.02-23.14: Comparison of At-Rest Soil Pressure to SSI Soil Pressure (Groundwater Table Six Feet below Grade) UHS Basin South Wall

RAI 03.07.02-24

QUESTION:

Follow-up Question to RAI 03.07.02-15 (STP-NRC-100036)

UHS Basin and RSW Pump House:

1. 10CFR50, Appendix S requires that evaluation for SSE must take into account soil-structure interaction (SSI) effects and the expected duration of vibratory motion. In the response to Item 6 of RAI 03.07.01-15, the applicant has provided a table summarizing the frequencies at which transfer functions are calculated as well as the cut-off frequency used in the SSI analysis for various analysis cases including the lower bound (LB), best estimate (BE) and upper bound (UB) in-situ soil cases; LB, BE and UB backfill soil cases; the cracked concrete and de-bonded soil case. The selected cut-off frequency for the different analysis cases varies from a low of about 16 Hz to a high of 25 Hz. The applicant has stated that the lowest cut-off frequency of 16 Hz meets the ASCE 4-98 Section C3.3.3.4 recommended values.

   With respect to the selected frequency cut-off and frequencies of analysis, the staff needs the following information:

   a) Staff has not endorsed ASCE 4-98 Section C3.3.3.4 as acceptable criteria for selecting the cutoff frequency for the SSI analysis for detailed finite element model such as UHS Basin with cooling tower enclosure and RSW Pump House. The applicant is requested to provide comparisons of in-structure response spectra at some selected locations by increasing the frequency cut-off to a minimum of 33 Hz and using a SSI model capable of transmitting a frequency up to 33 Hz (refer to Follow-up Question to RAI 03.07.02-17) for all analysis cases considered demonstrating that cut-off frequencies used in the SSI analysis are acceptable. The staff needs this information to ensure that the selected cut off frequencies less than 33 Hz in SSI analysis will accurately or conservatively account for the expected frequency content of the SSE in the SSI analysis.

   b) In reviewing the tabulated SSI analysis frequencies, it is observed that some frequencies are excluded from the calculation of un-interpolated transfer functions in certain directions. For example, the frequency 14.16 Hz is not included in the z-response analysis for the mean soil case and 9.521 Hz is not included in the z-response analysis for the upper bound soil case. The applicant is requested to provide the basis for selecting the frequencies of analysis for calculating the un-interpolated transfer functions and excluding any frequencies from such calculations. The staff requires this information to ensure that the SSI analysis results are not adversely affected by any numerical instability that may be caused by large numbers of soil layers used in SASSI to model deep non-uniform soil site at the UHS/RSW Pump House.

RSW Piping Tunnel:

10CFR50, Appendix S requires that evaluation for SSE must take into account soil-structure interaction (SSI) effects and the expected duration of vibratory motion. In order to ensure that evaluation of RSW Piping Tunnel for SSE has appropriately taken into account SSI effects, the staff needs the following information:

1. In the response to Item 1 of RAI 03.07.02-15, the applicant has stated that a 2-D SSI analysis of the RSW tunnel has been performed to quantify the in-structure response of the tunnel. No details of this analysis have been provided. As such, the applicant is requested to describe in sufficient detail in the FSAR how the SSI analysis of the RSW tunnel has been performed. The description shall include the SSI methodology, figures showing the SSI model and boundary conditions, summary of the soil and structure properties, the input motion, etc. so the review can be completed.

2. In the response to Item 2 of RAI 03.07.02-15, the applicant has stated that simple manual calculations were used for the analysis and design of individual components of the RSW piping tunnel. For this analysis, the tunnel walls, slabs and base mat are considered as rigid elements, and seismic loads are calculated based on a ZPA of 0.21g. The applicant further states that the analysis did not include any model or soil springs; the seismic loads are applied in terms of dynamic soil pressures on the exterior walls, calculated as per ASCE 4-98 recommendations. Staff has not endorsed ASCE 4-98 recommended dynamic soil pressures for design of tunnel walls. As such, the applicant is requested to provide comparisons of the dynamic soil pressures on the RSW tunnel walls calculated using 2-D SSI model versus those of ASCE 4-98 to demonstrate that the design pressures are still bounding when the effects of kinematic interaction between tunnel structures and surrounding soils as well as the effects of structure-soil-structure interaction (SSSI) due to nearby heavy structures are considered.

RESPONSE:

The following provides the response to part 1b of this RAI. The response to the remaining parts of this RAI will be provided in a supplemental response by October 25, 2010.

UHS Basin and RSW Pump House:

1b) For the Soil-Structure Interaction (SSI) analyses of the Ultimate Heat Sink/Reactor Service Water Pump House, selection of frequencies of analyses is an iterative process. An initial set of frequencies is selected for analysis and run for all cases. The analyses are processed to calculate preliminary responses for use in model checking. A review of transfer functions at a number of nodes is performed to determine where additional frequencies are needed to improve or verify the interpolated transfer functions. Upon completion of the analysis of additional frequencies, the transfer functions are again reviewed. If the set of calculated frequencies produces transfer functions that result in interpolated values justified by the adjacent calculated values the combined analyses

"Tape 8" are used for the processing of peak accelerations, response spectra, nodal displacements, and element demands. Each of these results is also reviewed, comparing results at neighboring nodes and elements. If discrepancies are discovered, additional transfer function review is performed and additional frequencies are added as needed and the process repeats.

This process of transfer function and analysis result review is time and labor intensive and must be repeated for each of the 8 analysis cases and 3 directions of analysis for a total of 24 analyses. When an analysis case and direction is reviewed and approved, no additional changes are made without justification. If additional frequencies are needed for a soil case in one direction, it is not added to another direction of analysis or soil case unless the other direction or soil case also needs improvement at the frequency in question. As a result of this process, each soil case and direction of analysis may contain a different set of frequencies that were used to create the final analysis set used to produce SSI results. Although the set of frequencies of analysis are not the same in every case, this is not a result of excluding any calculated frequencies from analysis. There is no requirement that each direction of analysis contain the same set of frequencies as another.

No additional COLA revision is required as a result of this response.

RAI 03.08.01-4, Revision 1

QUESTION:

In FSAR Appendix 3H, Section 3H.1.6, "Site Specific Structural Evaluation," the applicant addressed the effect of increased maximum flood level (STP DEP T1 5.0-1) for STP units 3 & 4 on the design of the Reactor Building (RB). In this section the applicant stated that "the load due to the revised flood level on the RB is less than the ABWR Standard Plant RB seismic load, and hence it doesn't effect the Standard Plant RB structural design." The staff considers this evaluation to be very qualitative, and the evaluation does not adequately address all issues associated with increased flood level. Therefore, the staff requests the applicant to provide a quantitative evaluation considering all effects due to the increased flood level including wave effects, if any, potential loadings due to flow and drag, overall stability of the structure considering floatation, etc. Also, it is not understood why the factor of safety for foundation stability considering buoyant forces from design basis flood reported in Table 3H.1-23 of the ABWR Standard Plant is not considered affected by the increased flood level. The same issue applies to the site specific structural evaluation of the control Building presented in Section 3H.2.6, and factor of safety for foundation stability reported in Table 3H.2-5 of the ABWR Standard Plant.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-090136, dated September 15, 2009. This response is being revised as a result of response to RAI 03.04.02-11 which is being submitted concurrently with this response. This revised response completely supersedes the original response. The revised portions of the response are marked with revision bars.

The following is based on the Main Cooling Reservoir (MCR) embankment breach analysis results provided in Attachment 1 of letter U7-C-STP-NRC-090012, dated February 23, 2009 and the response to RAI 03.04.02-11:

- Maximum calculated water level near the safety-related structures is at elevation 38.8 ft. Design flood level is conservatively established at elevation 40 ft.
- Maximum hydrodynamic drag force due to flood water flow is 44 pounds per square foot of the projected submerged area.
- Hydrodynamic forces due to wind generated wave forces are as shown in Figure 3.4-1 provided in the response to RAI 03.04.02-11.
- Impact due to a 500 lbs floating debris traveling at 4.72 ft/sec shall be considered.

The plant grade is at elevation 34 ft. Considering design flood level of 40 ft, the out-of-plane load on the above grade exterior walls of the Reactor Building (RB) and Control Building (CB) under flooded condition will be due to the hydrostatic pressure, hydrodynamic force due to flood flow of 44 lb/ft^2, hydrodynamic forces due to wind generated waves as shown in Figure 3.4-1 provided in the response to RAI 03.04.02-11 and impact due to a 500 lbs floating debris traveling at 4.72 ft/sec. This load is only applicable to the portion above grade elevation of 34 ft. For the below grade portions of the exterior walls, under flooded condition, the walls will be subjected to an increase of static water pressure due to 7 ft (from ground water elevation of 33 ft to design basis flood level of 40 ft) of water head.

Impact on the above grade walls

Above grade exterior walls of the RB and CB are designed for tornado loading which includes tornado generated missiles. Referring to Table 5.0 of DCD, Tier 1, the maximum tornado wind speed is 483 km/h (~300 mph) and the tornado missile spectrum includes an 1800 kg (~4000 lbs) automobile with horizontal impact velocity of 169.05 km/h (i.e. 0.35 x 483 = 169.05 km/h) or about 154 ft/sec. The kinetic energy of this tornado missile is over 8,500 times the kinetic energy of a 500 lbs floating debris traveling at 4.72 ft/sec [i.e. $(4000/500)(154/4.72)^2 = 8516.2$]. Thus, by engineering judgment the design of above grade exterior walls of the RB and CB for tornado wind pressure due to a wind speed of 300 mph in conjunction with tornado generated missiles is considered to bound the design for flood loading in conjunction with impact loading due to a 500 lbs floating debris traveling at 4.72 ft/sec.

Referring to the revised response to RAI 03.08.01-7, being submitted concurrently with this response, the calculated out-of-plane shear and moment demand for exterior walls of the RB and CB due to induced loading from MCR breach and safe-shutdown earthquake, SSE, are as follows:

For Reactor Building:

- Calculated out-of-plane shear and moment demands due to MCR breach are 1.72 k/ft and 3.83 k-ft/ft, respectively.
- Out-of-plane shear and moment demands due to SSE are 3.03 k/ft and 15.16 k-ft/ft, respectively.

For Control Building:

- Calculated out-of-plane shear and moment demands due to MCR breach are 1.67 k/ft and 3.59 k-ft/ft, respectively.
- Out-of-plane shear and moment demands due to SSE are 2.16 k/ft and 9.13 k-ft/ft, respectively.

Impact on the below grade walls

The increase in the out-of-plane load on the exterior walls of the RB and CB under flooded condition will be equal to 7 ft of water head or 7x62.4 = 436.8 psf. Referring to DCD Tier 2 Figures 3H.1-11 and 3H.2-14, the minimum seismic lateral soil pressure considered for design of below grade exterior walls of the RB and CB is 39.26 kPa or 819.96 psf which exceeds the 436.8 psf due to flood.

Based on the above, the out-of-plane flood loading on the exterior walls of the RB and CB are enveloped by out-of-plane SSE loading and thus the exterior walls of the RB and CB are adequate for resisting the induced flood loads from MCR embankment breach.

Impact on the stability safety factors

The flood load (excluding buoyancy) is only applicable to the lower 7.9 ft (see Figure 3.4-1) of the above grade portion of the RB and CB and thus the total flood load on these two structures is substantially less than the total seismic load which will be based on SSE excitation of the entire structure. Therefore, the sliding and overturning stability is not impacted. The effect of flooding on flotation safety factors is addressed below:

Per DCD Tier 2 Tables 3H.1-23 and 3H.2-5, the flotation safety factors for the RB and CB are 2.43 and 1.42 respectively. These flotation safety factors are based on maximum ground water level being one foot below grade (i.e. elevation 33 ft). Considering design flood level of 40 ft, the increased buoyancy force will result in revised flotation safety factors of 2.24 and 1.3 for RB and CB, respectively. These revised flotation safety factors are acceptable since they exceed the required flotation safety factor of 1.1 in accordance with Standard Review Plan 3.8.5.

For COLA revision as a result of this response, please see response to RAI 03.08.01-7 Revision 1 which is being submitted concurrently with this response.

RAI 03.08.01-7, Revision 1

QUESTION:

Follow-up question to Question 03.08.01-4 (RAI 2962)

The staff reviewed the applicant's response to Question 03.08.01-4 addressing the evaluation of standard plant structures for the increased flood level and needs the following additional information to complete the review:

(1) The applicant's response compares the out-of-plane shear and moment demands due to flood pressure with those due to the seismic load. The applicant did not include in its response any description or explanation about how the out-of-plane shear and moment demand for flood load and seismic load were obtained for the evaluation. Therefore, the staff requests the applicant to provide a detailed description of how the representative wall elements for the reactor building (RB) and the control building (CB) were selected for the evaluation, and how the reported shear and moment demands for flood and seismic load were determined.

(2) In its evaluation for impact of increased flood level on sliding and overturning stability, the applicant considered only the flood load acting on the bottom 6 ft of the above ground portion of the RB and the CB excluding buoyancy, and made a qualitative statement that the flood load is substantially less than the seismic load. Please explain why sliding and overturning of the structures due to flooding need not consider the hydrodynamic loads and the buoyancy effects on the structures, and provide a quantitative evaluation of sliding and overturning stability due to flooding. Please also update the FSAR to reflect that sliding and overturning of the RB and the CB were evaluated for the increased flood load on these structures.

(3) The applicant's response revises the factors of safety due to floatation for the RB and the CB, which are different from the values reported in Tables 3H.1-23 and 3H.2-5 of the ABWR DCD and in revised FSAR Sections 3H.1.6 and 3H.2.6. However, the applicant's response does not include the revision to the above ABWR DCD tables. Because the values of the floatation safety factors reported in DCD Tables 3H.1-23 and 3H.2-5 are no longer valid for the STP Units 3 and 4, the applicant is requested to address the issue appropriately.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100018, dated January 14, 2010. This response is being revised as a result of response to RAI 03.04.02-11 which is being submitted concurrently with this response. This revised response completely supersedes the original response. The revised portions of the response are marked with revision bars.

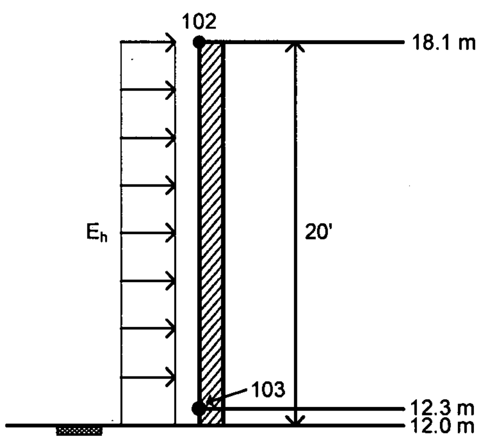
(1) Comparison of Out-of-Plane Shear and Moment Demands due to Flood and Seismic

The reported shear and moment demand comparison for the flood and seismic loadings in response to RAI 03.08.01-4 Revision 1 have been determined using the following parameters:

(a) Reactor Building:

Design of exterior walls of the Reactor Building under safe Shutdown Earthquake (SSE) loading will have to accommodate both in-plane and out-of-plane seismic loads. For the above grade walls, the design basis flood only affects the bottom 7.9 feet of the structure; thus, any in-plane load in the exterior walls due to design basis flood will be negligible in comparison to the seismic in-plane loads. However, when conservatively comparing the demand under seismic loading to the demand for flood loading, in-plane loads effect will be neglected and the comparison will be based on the demand for out-of-plane loads only. The parameters for determination of shear and moment demands for out-of-plane loads were as follows:

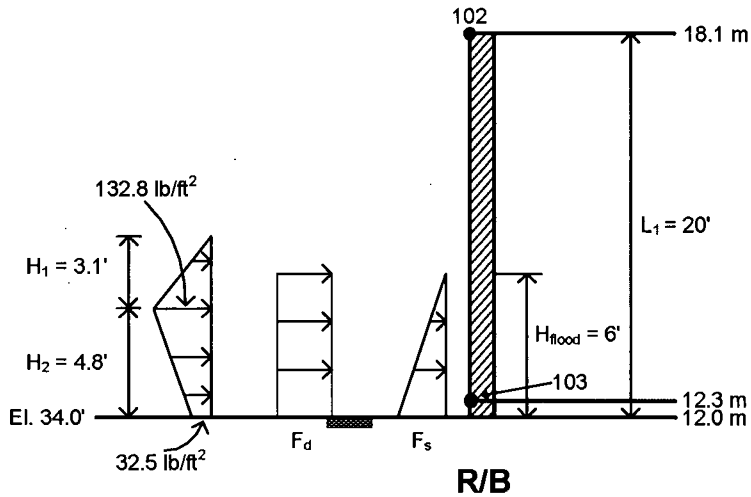
Seismic Loading:
Seismic acceleration at grade level = 0.47g (Conservative, see DCD Table 3A-23a, rigid zone acceleration for node 103)
Wall thickness = 4.3 ft
Wall weight = 150 lb/ft^3
Simply supported wall span = 20 ft (see figure below, conservatively the span is assumed to be from grade to node 102, this will yield a more critical shear demand comparison)
Applied out-of-plane load = 150 x 4.3 x 0.47 = 303.15 lb/ft^2
Calculated shear demand = 3.03 k/ft
Calculated moment demand = 15.16 k-ft/ft



R/B

Note: For nodes 102 and 103, see DCD Figure 3A-8

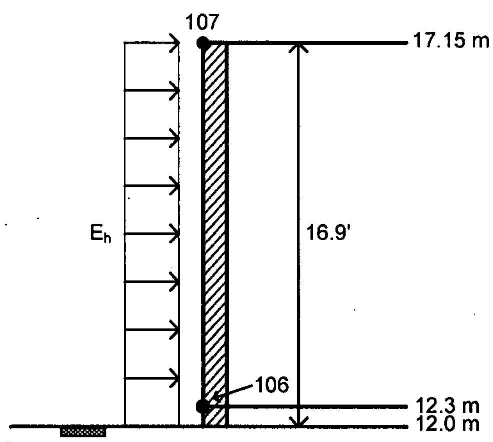
Flood Loading:
Simply supported span = 20 ft
Flood height = 6 ft (above grade)
Water density = 62.4 lb/ft^3
Hydrostatic head at grade (F_s) = 6 x 62.4 = 374.4 lb/ft^2
Hydrodynamic drag load due to flood water flow (F_d) = 44 psf (See response to RAI 03.08.01-4 Revision 1)
Hydrodynamic force due to wind generated waves (See figure below and response to RAI 03.04.02-11)
Calculated shear demand = 1.72 k/ft
Calculated moment demand = 3.83 k-ft/ft



(b) Control Building:

Similar to the Reactor Building, when conservatively comparing the demand under seismic loading to the demand for flood loading, in-plane loads effect will be neglected and the comparison will be based on the demand for out-of-plane loads only. The parameters for determination of shear and moment demands for out-of-plane loads were as follows:

Seismic Loading:
Seismic acceleration at grade level = 0.52g (Conservative, see DCD Table 3A-24, rigid zone acceleration for node 106)
Wall thickness = 3.28 ft
Wall weight = 150 lb/ft^3
Simply supported wall span = 16.9 ft (see figure below, conservatively the span is assumed to be from grade to node 107, this will yield a more critical shear demand comparison)
Applied out-of-plane load = 150 x 3.28 x 0.52 = 255.84 lb/ft^2
Calculated shear demand = 2.16 k/ft
Calculated moment demand = 9.13 k-ft/ft

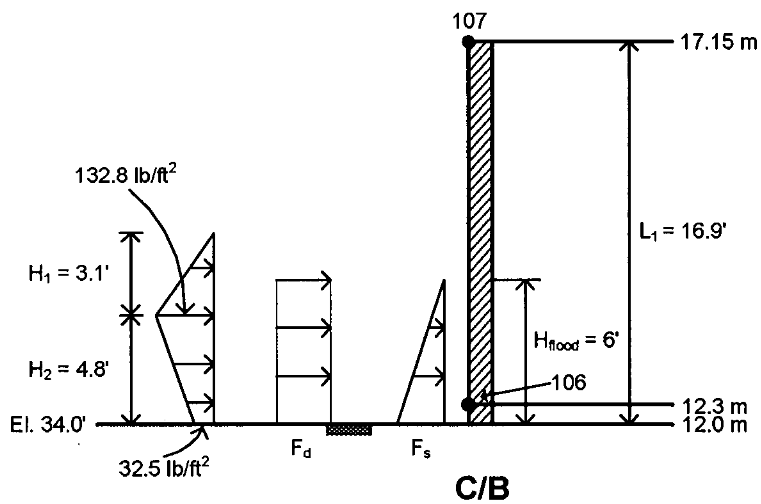


C/B

Note: For nodes 106 and 107, see DCD Figure 3A-27

Flood Loading:

Simply supported span = 16.9 ft
Flood height = 6 ft (above grade)
Water density = 62.4 lb/ft^3
Hydrostatic head at grade (F_s) = 6 x 62.4 = 374.4 lb/ft^2
Hydrodynamic drag load due to flood water flow (F_d) = 44 psf (See response to RAI 03.08.01-4 Revision 1)
Hydrodynamic force due to wind generated waves (See figure below and response to RAI 03.04.02-11)
Calculated shear demand = 1.67 k/ft
Calculated moment demand = 3.59 k-ft/ft



(2) Impact of Increased Flood Level on Sliding and Overturning Stability:

Stability requirements for the Reactor and Control Buildings are specified in Sections 3H.1.4.5 and 3H.2.4.5 of the ABWR DCD Tier 2, respectively. These requirements are consistent with Standard Review Plan (SRP) Section 3.8.5.

Referring to SRP Section 3.8.5 as well as the above-noted DCD Tier 2 requirements, the following load combinations and acceptance criteria are applicable:

"………., the combinations used to check against sliding and overturning attributable to earthquakes, winds, tornadoes and against flotation because of floods are acceptable if found to be in accordance with the following:

A. D + H + E
B. D + H + W
C. D + H + E'
D. D + H + Wt
E. D + F'

Where D, E, W, E', and Wt are as referenced in Subsection II.3 of SRP Section 3.8.4, where H is the lateral earth pressure, and F' is the buoyant force of the design basis flood. Justification should be provided for including live loads or portions thereof in these combinations.

Structural Acceptance Criteria. For the loading combinations referenced in the first paragraph of Subsection II.3 of this SRP section, the allowable limits that constitute the acceptance criteria are referenced in Subsection II.5 of SRP Section 3.8.1 for the containment foundation and in Subsection II.5 of SRP Section 3.8.4 for all other foundations. In addition, for the five other load combinations in Subsection II.3 of this SRP section, the factors of safety against overturning, sliding, and flotation are acceptable if found to be in accordance with the following:

Minimum Factors of Safety

For Combination	Overturning	Sliding	Flotation
a. ----------------------	1.5	1.5	---
b. ----------------------	1.5	1.5	---
c. ----------------------	1.1	1.1	---
d. ----------------------	1.1	1.1	---
e. ----------------------	---	---	1.1 "

As can be seen from the above, when considering design basis flood, neither SRP Section 3.8.5 nor DCD require checking sliding and/or overturning. Nonetheless, even if one were to check sliding and overturning due to unbalanced forces on the Reactor and Control Buildings due to the design basis flood (only 6 feet above grade), the unbalanced forces due to design basis flood in comparison to the unbalanced loads due to seismic SSE will be quite negligible such that even with increased buoyant force due to additional 7 feet of water (from ground water elevation of 33 ft to design basis flood level of 40 ft), the seismic load combination will remain as the controlling load combination for sliding and overturning of the Reactor and Control Buildings.

As noted in our response to RAI 03.08.01-4, as a result of 7 feet increase in the elevation of design basis flood, the flotation factors of safety for the Reactor and Control Buildings will reduce to 2.24 and 1.3, respectively. These revised safety factors are acceptable since they exceed the required flotation safety factor of 1.1 in accordance with the DCD and SRP Section 3.8.5.

(3) Update of Tables 3H.1-23 and 3H.2-5:

The COLA will be revised with the following site-specific supplemental information from DCD Tier 2, Subsections 3H.1.6, 3H.2.6 and Table 3H.1-23 and 3H.2-5 as revised below:

a. Section 3H.1.6

As documented in Subsection 3.4, the STP 3 & 4 site has a design basis flood elevation that is 182.9 cm above grade. This results in an increase in the flood level over what was used in the ABWR Standard Plant, however the load due to the revised flood level, including hydrodynamic drag load due to flood water flow and hydrodynamic load due to wind generated wave action as described in Section 3.4.2, on the exterior above and below grade RB walls is less than the ABWR Standard Plant RB seismic load. The design of above grade RB exterior walls for design basis tornado loading per Table 5.0 of DCD, Tier 1, including tornado generated missiles bounds the design for flood loading including impact due to floating debris. H~~h~~encre it doesn't affect the Standard Plant RB structural design.

The factor of safety against floatation has been calculated and is shown in revised Table 3H.1-23.

Table 3H.1-23 Factors of Safety for Foundation Stability*

Load Combination	Overturning		Sliding		Floatation	
	Req'd.	Actual	Req'd.	Actual	Req'd.	Actual
D + F'					1.1	~~2.43~~2.24
D + Lo + F + H+ Ess	1.1	490	1.1	1.11		

Here:
F = Buoyant Forces from Design Ground Water (0.61m Below Grade)
F' = Buoyant Forces from Design Basis Flood (~~0.3m Below~~ 1.83m Above Grade)
H = Lateral Soil Pressure
Lo = Live Load Acting During an Earthquake (Zero Live Load is Considered).
Ess = SSE Load
D = Dead Load

b. Section 3H.2.6

As documented in Subsection 3.4, the STP 3 & 4 site has a design basis flood elevation that is 182.9 cm above grade. This results in an increase in the flood level over what was used in the ABWR Standard Plant, however the load due to the revised flood level, including hydrodynamic drag load due to flood water flow and hydrodynamic load due to wind generated wave action as described in Section 3.4.2, on the exterior above and below grade CB walls is less than the ABWR Standard Plant seismic load. The design of above grade CB exterior walls for design basis tornado loading per Table 5.0 of DCD, Tier 1, including tornado generated missiles bounds the design for flood loading including impact due to floating debris. H~~h~~ence it does not affect the Standard Plant CB structural design.

The factor of safety against floatation has been calculated and is shown in revised Table 3H.2-5.

Table 3H.2-5 Stability Evaluation–Factors of Safety

Load Combination	Overturning Required	Overturning Actual	Sliding Required	Sliding Actual	Flotation Required	Flotation Actual
D+F’	–	–	–	–	1.1	~~1.42~~1.30
D+F+H+W	1.5	2.79	1.5	2.74	–	–
D+F+H+Wt D+Lo+F+H’+E’**	1.1 1.1	2.66 123*	1.1 1.1	2.69 1.14	– –	– –

* Based on the energy technique
** Zero live load is considered.
F’ = Buoyant Forces from Design Basis Flood (1.83m Above Grade)

RAI 03.08.01-9

QUESTION:

Follow-up to Question 03.08.01-6

In its response to Question 03.08.01-6, the applicant addressed some of the issues regarding the watertight doors. However, additional information is needed to completely address all of the issues pertaining to the design of the watertight doors. In order for the staff to complete its review, the applicant is requested to provide the following additional information:

1. In Section 2 of the response, the applicant provided a sketch that shows the location of the watertight door between the Control building and the Radwaste Building Access Corridor. However, the applicant did not include the sketch in the FSAR mark-up provided with the response. Therefore, the applicant is requested to include the sketch in the FSAR to clearly identify locations of all seismic category I watertight doors.

2. In Section 3(a) of the response, the applicant provided loadings and loading combinations for design of watertight doors considering flooding. The staff needs the following clarifications for the loads and load combinations provided in the response:

    a. Since ANSI/AISC N690 and ACI 349 do not specifically address flood loads, please explain how the flood loads and the loading combinations, including the load factors used in loading combinations involving flood load, were determined with reference to applicable industry codes and standards. Please include in FSAR Section 3H.6.4.3.3.4, "Extreme Environmental Flood (FL)," a description of the various components of flood load, e.g., hydrostatic load, hydrodynamic load, impact load from debris transported by lood water, etc., and the orresponding design values used.

    b. The applicant defined pressure load 'P' as hydrostatic or differential pressure, and used t in several loading combinations. Please explain why only pressure load 'P' need to be onsidered for design of watertight doors, and not the other components of FL, e.g., hydrodynamic load and load from debris transported by flood.

3. In Section 3(b) of the response, the applicant stated that the doors will be designed in accordance with AISC N690. Since it is not clear which version of ANSI/AISC N690 was used by the applicant, please confirm that the version of the specification used is the same as that referenced in SRP 3.8.4 and update FSAR accordingly, or provide justification for using a different version.

4. In response to the staff's question regarding design and analysis procedure used for the watertight doors, the applicant stated in Section 3(c) of the response that "the design of the door will be performed in accordance with the requirements of SRP Section 3.8.4." SRP 3.8.4 provides general guidance and acceptance criteria for analysis and design

procedure of concrete and steel category I structure. Merely referencing the SRP does not provide any information about the analysis and design procedure used by the applicant. Therefore, the applicant is requested to include in the FSAR a description of the analysis and design procedure including how seismic loads are determined for the watertight doors.

5. In response to the staff's question regarding testing and in-service inspection of the watertight doors, the applicant stated in Section 3(f) of the response, and the FSAR mark-up included in the response, that the watertight doors will allow slight seepage during an external flooding in accordance with criteria for Type 2 closures in U.S. Army Corps of Engineers (COE) EP 1165-2-314. The applicant also stated that this criterion will be met under hydrostatic loading of 12 inches of water above the design basis flood level. The applicant further stated that the water retaining capability of the doors will be demonstrated by qualification tests that shall not allow leakage more than 1/10 gallon per linear foot of gasket when subjected to the specified head pressure plus a 25% margin for one hour. The applicant did not provide in the response any information regarding in-service inspections of the watertight doors. In order for the staff to assess adequacy of the watertight doors and their availability when needed, please provide the following additional information:

    a. The allowable leakage of 1/10 gallon per linear foot of gasket per hour may potentially allow ingress of significant amount of water over time. Please provide justification why this leakage is considered to meet criterion for Type 2 closure, which is defined to form essentially dry barriers or seals, and the basis for the underlying assumption that such leakage will not compromise functionality of any safety related commodity or any other design basis.

    b. Since hydrostatic pressure on the door may help in providing a seal for the door, please explain why testing these doors against the maximum water pressure only is adequate, and will envelope performance of the seals during lower hydrostatic pressure.

    c. Since the applicant did not include in its response any information about the in-service surveillance programs for the watertight doors, and corresponding FSAR update, please explain how availability of the normally open watertight doors during a flooding event is ensured considering that these doors will need to be closed upon indication of an imminent flood.

6. In Section 6 of the response, the applicant states that the access doors between the Reactor Building (RB) and Control building (CB) are not required to be watertight since both buildings are separately protected from design basis flood, and the gap between the two buildings will be sealed using the detail shown in Figure 03.08-04-15A, which is attached to the response to RAI 03.08.04-15 (see STPNOC letter U7-C-STP-NRC-090160 dated October 5, 2009). The above referenced Figure provides only a conceptual detail of a joint seal between the buried Reactor Service Water (RSW)

tunnels, and the RSW Pump House and the Control Buildings. In its response to a subsequent follow-up question 03.08.04-25 for the above referenced joint seal, the applicant provided additional design criteria for the seals to accommodate differential movements across the seal, and explained that because of the low rate with which groundwater can flow through the seal if it were to fail in any particular location, the in-leakage of groundwater is a housekeeping issue and not a safety concern. Since the seals for the gaps between the RB and the CB are credited to prevent ingress of flood water into these buildings and provide protection to safety related commodities against flooding, reference to the joint seals used for the RSW tunnels does not adequately address the issue of ingress of flood water and potential damage to safety related components. Therefore, the applicant is requested to include in the FSAR a description of the seal between the RB and the CB including information about seismic classification, performance demand, qualification, and in-service inspection of the seal to demonstrate that the seals will be capable of preventing flood water from entering these buildings under all postulated design basis loading conditions.

The staff needs the above information to conclude that the watertight doors are designed for appropriate loads and load combinations, pertinent design information per guidance provided in SRP 3.8.4 are included in the FSAR, and there is reasonable assurance that the normally open watertight doors will be available during a flooding event.

RESPONSE:

1. The watertight door between the Control Building and the Radwaste Building Access Corridor shown in response to RAI 03.08.01-6, submitted with STPNOC letter U7-C-STP-NRC-100018, dated January 14, 2010, was deleted in the revised response to RAI 03.08.01-6, submitted with STPNOC letter U7-C-STP-NRC-100154 dated June 29, 2010. Therefore, the sketch provided in response to RAI 03.08.01-6 was removed in the revised response to RAI 03.08.01-6 and no FSAR revision is required to include this door.

2a. It is acknowledged that the load combinations in ANSI/AISC N690 and ACI 349 do not specifically address flood loads. However, Section R9.2.7 of the Commentary to ACI 349-97 states that:

"Apart from the extreme environmental loads generated by the safe shutdown earthquake and by the design basis tornado, other extreme environmental loads may also be required for the plant design. Examples of such loads are those induced by flood, aircraft impact, or an accidental explosion.

These environmental loads should be treated individually in a manner similar to the loads generated by the design basis tornado in determining the required strength according to the equations in Section 9.2.1. Abnormal loads are not considered concurrently with the above extreme environmental loads."

The controlling flood at STP 3&4 site is due to the Main Cooling Reservoir dike breach. This load is considered to be an extreme environmental load, and therefore is treated as described in Section 9.2.7 of ACI 349-97. Consistent with Section 9.2.7 of ACI 349-97, the load factors are taken as 1.0.

The COLA markup provided with RAI 03.04.02-6, submitted with STPNOC letter U7-C-STP-NRC-100154 dated June 29, 2010 included the following load combination for flooding:

$1.6S = D + P + E'$

In this load combination P included the load due to the flood. The load combinations will be revised as follows:

$S = D + W + P_o$
$1.6S = D + E' + P_o$
$1.6S = D + W_t + P_o$
$1.6S = D + FL + P_o$

Where:

S = Normal allowable stresses as defined in AISC N690
D = Dead loads
P_o = Normal Operating Differential Pressure
E' = Loads generated by SSE, per Sections 3H.1 and 3H.2.
FL = Design basis extreme flood loads, including the hydrostatic load due to flood elevation at 40 ft MSL, the associated drag effects of 44 psf, hydrodynamic load due to wind-generated wave action per Figure 3.4-1, and impact due to floating debris per Section 3.4.2 (Figure 3.4-1 and revised Section 3.4.2 are included in response to RAI 03.04.02-11, which is being submitted concurrently with this response).
W = Normal wind loads, per DCD Sections 3H.1 and 3H.2
W_t = Tornado loads per DCD Sections 3H.1 and 3H.2, including wind velocity pressure W_w, differential pressure W_p, and tornado-generated missiles (if not protected) W_m

2b. With the revised load combinations and load definitions provided in 2a. above the question related to definition of P and flood loads is answered. Drag load and load from debris transported by flood load is considered, as discussed above.

3. For the site-specific Diesel Generator Fuel Oil Storage Vault the applicable version of ANSI/AISC N690 is 1994 with Supplement 2 in accordance with the Standard Review Plan (SRP) Section 3.8.4, Revision 2 (the revision applicable to site-specific structures). COLA Table 1.8-21a will be revised to include this revision of the Code for site-specific application, as shown in the response to

RAI 03.08.04-33, which is being submitted concurrently with this response. For the Reactor and Control Building, the applicable version of ANSI/AISC N690 is 1984, as listed in DCD Table 1.8-21. These versions will be used in the design of the doors, as applicable.

4. The watertight doors will be designed by vendors in accordance with specific requirements given in the procurement specification. The procurement specification will include the requirement that the detailed analysis and design comply with the requirements of applicable revision of SRP Section 3.8.4 and AISC N690. The seismic loads will be determined using the applicable response spectra. The method of analysis for evaluation of seismic and other reactor building vibratory loadings, if applicable, will be the static equivalent method as described in DCD Section 3.7.3.8.1.5.

5a. The criterion for Type 2 closure is to allow slight seepage during the hydrostatic pressure conditions of flooding. Specifically, the requirements for Type 2 Closures are defined in U.S. Army Corps of Engineers (COE) EP 1165-2-314 Section 701.1.2 and requires that the closure:

"shall form essentially dry barriers or seals, allowing only slight seepage during the hydrostatic pressure conditions of flooding to the RFD."

There are less than 1000 linear feet of gasket material for all the watertight doors used for protection against external flooding. A leakage rate of 1/10 gallon per linear foot of gasket per hour equates to 100 gallons/hour or 0.006 m^3/min. The allowable leakage of 1/10 gallon per linear foot of gasket per hour is far less than the 1.34 m^3/min accepted for internal flooding in Reactor Building elevation 1F in DCD Section 3.4.1.1.2.1.4 and the 12.0 m^3/min accepted for internal flooding in the Control Building in DCD Section 3.4.1.1.2.2 due to internal pipe leakage. The safety related equipment potentially subjected to external flooding is protected by curbs and raised equipment pads, similar to the safety related equipment potentially subjected to internal flooding.

5b. During the test, the hydrostatic head will be raised at a rate not more than 1 ft/min to a level of 25% higher than the flood level. Any leaks that occur during this time will be detected and if the leakage rate begins to diminish as the hydrostatic head increases, the assembly will be tested at a lower hydrostatic head. This requirement is added to the COLA markup provided in the revised response to RAI 03.04.02-6, being submitted concurrently with this response.

5c. The revised responses to RAI 03.04.02-6 and RAI 19-30 (submitted with STPNOC letter U7-C-STP-NRC-100119 dated May 27, 2010) now state that all doors that protect against the design basis flood will be normally closed. For requirements pertaining to inspection and maintenance, see the response to

RAI 03.04.01-6 submitted with STPNOC letter U7-C-STP-NRC-090045 dated May 13, 2009.

6. The seals between the Reactor Building and the Control Building below the design basis flood level will be made using a polyurethane foam impregnated with a waterproof sealing compound. The seals will be tested to be watertight when subjected to the maximum anticipated hydrostatic head at movements of +/-25% of the designed gap size. The lowest required watertight seal is in the slab at nominal elevation 4.8m (the lowest elevation of the Clean Access Corridor between the Reactor Building and Control Building) and the hydrostatic head associated with this seal is not anticipated to exceed 35 ft. The seals used to protect the safety-related buildings against external water entry are classified as seismic category I with respect to their ability to remain in-place to stop significant water leakage into the safety-related buildings during and after a seismic event. While the gap size is determined based on the displacement under a Safe Shutdown Earthquake (SSE) load, similarly to the joints discussed in RAI 03.08.04-25, submitted with STPNOC letter U7-C-STP-NRC-100108 dated May 13, 2010, in-leakage of groundwater through a degraded flexible filler material due to an SSE event is a housekeeping issue and not a safety concern. Movements of +/-25% of the gap size will envelope any expected displacements anticipated under normal settlement loading. This will show the material is capable of being watertight after the effects of long-term settlement and tilt, as well as during normal operating vibratory loads, such as SRV actuation. Although this will provide margin to accommodate additional differential displacements from the majority of the movements from short duration extreme environmental loading, such as SSE and tornado, the seals need not be designed to be watertight during the differential displacements from these extreme environmental loadings. Leakage during local seal failure due to extreme environmental loading events will be significantly less than the 1.34 m^3/min accepted for flooding in Reactor Building elevation 1F in DCD Section 3.4.1.1.2.1.4 and the 12.0 m^3/min accepted for flooding in the Control Building in DCD Section 3.4.1.1.2.2 due to internal pipe leakage. An in-service inspection program will ensure that the seals do not significantly degrade during normal plant operation and after being subjected to an extreme environmental loading event. This will ensure that the seals adequately protect safety-related equipment from significant leakage of water into the Reactor Building and Control Building. The requirements discussed above are added to the COLA markup provided in response to RAI 03.04.02-6.

The COLA markups resulting from this response are included in the revised COLA markup included in the revised response to RAI 03.04.02-6, being submitted concurrently with this response. No additional COLA revision is required as a result of this response.

RAI 03.08.01-10

QUESTION:

Follow-up to Question 03.08.01-7

In response to Question 03.08.01-7, Section (1), the applicant provided details of how the out-of-plane shear and moment demands for flood and seismic loads were determined. The staff notes that the applicant in its response did not consider loading due to floating debris for computing shear and moment demands for flood. Also, the applicant implicitly used the loading combination for flood load as shown in FSAR Section 3H.6.4.3.4.3. This loading combination is not included in ACI 349, "Code Requirements for Nuclear Safety Related Concrete Structures," as referenced in SRP 3.8.4. Further, computations of shear and moment demands due to flood loading for the RB and CB walls appear to be incorrect for the assumed boundary conditions for the wall sections. Therefore, in order for the staff to be able to conclude that the ABWR standard plant structures are capable of withstanding the site specific flood load, the applicant is requested to provide the following additional information:

1. Please include the effect of debris in flood water in the evaluation of representative wall elements of the Reactor Building (RB) and the Control Building (CB) for design basis flood. The staff notes that in its response to Question 03.08.04-22, the applicant had considered loading due to debris in flood water by considering the unit weight of flood water to be 80 pounds per cubic foot (pcf). Please provide justification for assumed debris loading with reference to industry standards and codes, as applicable.

2. Please provide the basis for the loading combination used for flood loading with reference to applicable industry codes and standards.

Please review the computations for shear and moment demands due to flood for RB and CB wall sections included in the response, and correct them, as needed.

RESPONSE:

1) In order to account for impact of floating debris, guidance provided in Section C5 of the Commentary to ASCE 7-05 was used. Based on this, impact due to a floating debris weighing 500 lbs and traveling at maximum flood water velocity of 4.72 ft/sec is considered. For evaluation of effect of floating debris, please see RAI 03.08.01-4 Revision 1 response, being submitted concurrently with this response.

   The flood water density, considering maximum sediment concentration, is 63.85 pounds per cubic foot (pcf) per COLA Section 2.4S.4.2.2.4.3. The density of 80 pcf noted in response to RAI 03.08.04-22 was a conservatively assumed value. This value is being revised to 63.85 pcf in the revised response to RAI 03.08.04-22, being submitted concurrently with this response.

Evaluations for effect of design basis flood for the RB and CB are reported in the RAI 03.08.01-7 Revision 1 and RAI 03.08.01-4 Revision 1 response. In these evaluations, a water density of 62.4 pcf instead of 63.85 pcf is used, which is justified based on the following:

- Per COLA Section 2.4S.2.2, the maximum calculated flood elevation due to MCR embankment breach is 38.8 ft MSL. The design basis flood level is conservatively established as 40 ft MSL.
- As stated above, per COLA Section 2.4S.4.2.2.4.3, the flood water density considering maximum sediment concentration is 63.85 pcf.
- Based on the above, and considering STP finished grade of 34 ft MSL, the maximum flood water head is 306.5 psf [i.e. (38.8-34)63.85 = 306.5 psf]. This water head of 306.5 psf is less than the water head of 374.4 psf used in the response to RAI 03.08.01-7.

2. The load combination used for flood loading is based on requirements of Section 9.2.7 of ACI 349-97 shown below:

   "**9.2.7** If resistance to other extreme environmental loads such as extreme floods is specified for the plant, then an additional load combination shall be included with the additional extreme environmental load substituted for ***Wt*** in Load Combination 5 of 9.2.1"

3. The reported shear and moment demands in the original response to RAI 03.08.01-7 were conservatively calculated considering a uniform loading of 418.4 psf for the entire flood height of 6 ft (i.e. 374.4 psf due to 6 ft water head plus 44 psf due to drag load due to flood water). Please see RAI 03.08.01-7 Revision 1 response for the latest calculated shear and moment demands due to flood loading, including hydrodynamic loads due to wind generated waves.

No additional COLA revision is required as a result of this response.

RAI 03.08.04-18, Revision 1, Supplement 1

<u>QUESTION:</u>

Follow-up to Question 03.08.04-2 (RAI 2964)

The applicant's response to Question 03.08.04-2 states that the Radwaste Building (RWB) will be designed in accordance with the requirements of RG 1.143, Revision 2. The applicant also discussed the design criteria for this building for seismic category II/I evaluation. In order for the staff to conclude that the Radwaste Building design meets the requirements of RG 1.143, and also meets the requirement in ABWR DCD Section 3.7.2.8, item (3), the FSAR needs to include sufficient design information for the building to demonstrate that the design meets the pertinent design criteria. Guidance provided in SRP Section 3.8.4 may be used for providing such information. Therefore, the applicant is requested to provide design information for the RWB in the FSAR that includes more detailed description of the structure; applicable codes, standards and specifications; loads and load combinations including live loads, seismic loads, thermal loads, flood loads, tornado loads, lateral soil pressure, etc.; design and analysis procedures; structural acceptance criteria; materials and quality control; design of critical sections, stability evaluation, etc.

<u>SUPPLEMENTAL REVISED RESPONSE:</u>

The Revision 1 response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100124, dated June 2, 2010 which provided the analysis and design results for the Radwaste Building. The following supplemental response provides additional information for the flooding loads for Category II/I evaluation. This additional information is based on the response to RAI 03.04.02-11, being concurrently submitted with this response.

The COLA mark-up provided for Section 3H.3.5.3 with the Revision 1 response to this RAI will be revised as shown below to add the hydrodynamic effect due to the wind-generated wave action. This revised mark-up completely supersedes the mark-up provided for this section with Revision 1 response to this RAI.

Please also refer to the response to RAI 03.04.02-11.

3H.3.5.3 Seismic II/I Evaluation

The seismic II/I evaluation for the RWB is performed to ensure that the RWB will not collapse on the nearby Category I structures. The structure is conservatively designed to remain elastic for this evaluation. The earthquake input used at the foundation level is the envelope of 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum due to site-specific SSE that is determined from an SSI analysis which accounts for the impact of the nearby Reactor Building (RB). In this SSI analysis, five interaction nodes at the depth corresponding to the bottom elevation of the RWB foundation are added to the three dimensional SSI model of the RB. These five interaction nodes correspond to the four corners and the center of the RWB foundation. The average response of these five interaction nodes is enveloped with the 0.3g RG 1.60 spectra to determine the SSE input at the foundation level.

For tornado parameters, including the missiles, the same parameters as those defined in DCD Tier 1 Table 5.0 are used. For flood, the extreme flood level of 40 ft (12.2 meters) MSL ~~with a drag force of 44 psf~~ is used, which is caused by the Main Cool~~ant~~ing Reservoir dike breach. The evaluation requirements for this flood, including hydrodynamic and flooding debris loading, are included in Section 3.4.2.

The II/I stability evaluations for sliding and overturning are performed using the site-specific SSE and other site-specific parameters such as soil properties.

RAI 03.08.04-22, Revision 1

QUESTION:

Follow-up to Question 03.08.04-12 (RAI 2965)

The applicant's response to Question 03.08.04-12 refers to the response submitted for RAI 03.07.01-13 (see letter U7-C-STP-NRC-090112, dated August 20, 2009). However, a review of the FSAR subsections identified in that response reveals that the response provided only a definition of these loads, and the thermal, hydrostatic and lateral soil pressure load values are not provided. Therefore, the applicant is requested to include in the FSAR the values of the thermal, hydrostatic and lateral soil pressure loads that are used in the analysis.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100036, dated February 10, 2010 which provided the loading information for the design of Reactor Service Water (RSW) Piping Tunnel and Ultimate Heat Sink (UHS)/Reactor Service Water (RSW) Pump House. This revised response provides additional information for the flooding loads. This additional information is based on the response to RAI 03.04.02-11, being concurrently submitted with this response. This revised response completely supersedes the original response. The revisions are marked in the margin.

Please also refer to the response of RAI 03.04.02-11.

Thermal Loads:

The RSW piping tunnels are not subjected to any thermal loads. The thermal loads applied to the UHS/RSW Pump House finite element model are calculated as follows:

Notation:

T_c = reference concrete placement temperature
T_i = inside temperature
T_o = outside temperature
t = thickness of section (wall/slab)

Thermal gradient load = $(T_i - T_o) / t$

Thermal axial load = $[(T_i + T_o) / 2] - T_c$

Thermal gradient loads and thermal axial loads are applied to the finite element model for six (6) separate thermal conditions.

The following temperature values are applicable to all six (6) thermal conditions:
60 °F reference concrete placement temperature
70 °F soil temperature
90 °F pump house inside air temperature

The basin water temperatures and the outside air temperatures for the six thermal conditions are as follows:

(1) Winter – Accident Basin Water Temperature:

95 °F basin water temperature
24 °F outside air temperature

This thermal condition maximizes the winter thermal gradient across the basin walls.

(2) Winter - Minimum Basin Water Temperature:

50 °F basin water temperature
24 °F outside air temperature

This thermal condition maximizes the thermal axial contraction of the basin walls.

(3) Winter – Typical Operating Temperature:

55 °F basin water temperature
45 °F outside air temperature

This thermal condition is applicable only for basin basemat and basin walls below 71 ft maximum water level with ACI 350-01 durability factors. Per Section 9.2.7 of ACI 350-01, estimation of contraction, expansion, and temperature change should be based on realistic assessment of such effects occurring in service. Section R.9.2.7 of ACI 350-01 specifically states that the term "realistic assessment" is used to indicate the most probable values rather than the upper bound values.

(4) Summer - Accident Basin Water Temperature:

95 °F basin water temperature
90 °F outside air temperature

This thermal condition maximizes the thermal axial expansion of the basin walls.

(5) Summer – Minimum Basin Water Temperature:

60 °F basin water temperature
90 °F outside air temperature

This thermal condition maximizes the summer thermal gradient across the basin walls.

(6) Summer – Typical Operating Temperature:

95 °F basin water temperature
90 °F outside air temperature

This thermal condition is applicable only for basin basemat and basin walls below 71 ft maximum water level with ACI 350-01 durability factors. Conservatively, the summer accident temperatures are considered as the typical summer operating temperatures.

Design Basis Flood Load:

The design basis flood level is conservatively established as 40.0 ft MSL, in accordance with Subsections 2.4S.2.2 and 3H.6.4.2.3. The flood water unit weight, considering maximum sediment concentration, is 63.85 pcf per Section 2.4S.4.2.2.4.3. The design requirements for this flood, including hydrostatic, hydrodynamic, and floating debris loading, are included in Section 3.4.2.

Hydrostatic Loads:

This load is only applicable to the UHS/RSW Pump House. For all load combinations in the finite element model analysis of UHS/RSW Pump House, the hydrostatic load due to water inside the basin is conservatively calculated considering the maximum water height of 71 ft above the top of the UHS basin basemat. The maximum hydrostatic pressure is 4.43 ksf at the top of UHS basin basemat elevation.

Lateral Soil Pressure:

Lateral soil pressures used for design of UHS/RSW Pump House and RSW Piping Tunnels (Figures 3H.6-41 through 3H.6-44) and stability evaluations of the UHS/RSW Pump House (Figures 3H.6-45 through 3H.6-50) have been provided as part of Supplement 2 response of RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230 dated 12/30/2009).

As a result of this response, COLA Part 2, Tier 2, Sections 3H.6.4.3.1.4 through 3H.6.4.3.1.6, 3H.6.4.3.3.3, 3H.6.4.3.3.4, and 3H.6.4.3.4.3 will be revised and Section 3H.6.4.3.4.4 will be added as shown below:

3H.6.4.3.1.4 Lateral Soil Pressures (H)

Lateral soil pressures are calculated using the following soil properties.

- Unit weight (moist):..120 pcf (1.92 t/m3)
- Unit weight (saturated): ....140 pcf (2.24 t/m3)
- Internal friction angle:..30°
- Poisson's ratio (above groundwater)...................................0.42
- Poisson's ratio (below groundwater)....................................0.47

The calculated lateral soil pressures are presented in figures as indicated:

- Lateral soil pressures for design of UHS/RSW Pump House: Figures 3H.6-41 through 3H.6-43.
- Lateral Soil pressures for design of RSW Piping Tunnels: Figures 3H.6-44.
- Lateral soil pressures for stability evaluation of UHS/RSW Pump House: Figures 3H.6-45 through 3H.6-50.

3H.6.4.3.1.5 Thermal Loads (To)

~~Internal moments and forces caused by temperature distribution.~~

The RSW piping tunnels are not subjected to any thermal loads. Thermal gradient loads and thermal axial loads are applied to the UHS/RSW Pump House finite element model for six (6) separate thermal conditions.

The following temperature values are applicable to all six (6) thermal conditions:

- Reference concrete placement temperature.....................60°F
- Soil temperature...70°F
- Pump house inside air temperature...............................90°F

The basin water temperature and the outside air temperature for the six (6) thermal conditions are as follows:

(1) Winter – Accident Basin Water Temperature

- Basin water temperature...95°F
- Outside air temperature...24°F

(2) Winter – Minimum Basin Water Temperature

- Basin water temperature..50°F
- Outside air temperature..24°F

(3) Winter – Typical Operating Temperatures

- Basin water temperature..55°F
- Outside air temperature..45°F

This thermal condition is applicable only for the basin basemat and basin walls below the 71 ft maximum water level with ACI 350-01 durability factors. Per Section 9.2.7 of ACI 350-01, estimation of contraction, expansion, and temperature change should be based on realistic assessment of such effects occurring in service. Section R.9.2.7 of ACI 350-01 specifically states that the term "realistic assessment" is used to indicate the most probable values rather than the upper bound values.

(4) Summer – Accident Basin Water Temperature

- Basin water temperature..95°F
- Outside air temperature..90°F

(5) Summer – Minimum Basin Water Temperature

- Basin water temperature..60°F
- Outside air temperature..90°F

(6) Summer – Typical Operating Temperatures

- Basin water temperature..95°F
- Outside air temperature..90°F

This thermal condition is applicable only for the basin basemat and basin walls below the 71 ft maximum water level with ACI 350-01 durability factors. Conservatively, the summer accident temperatures are considered as the typical summer operating temperatures.

3H.6.4.3.1.6 Hydrostatic Loads (F)

~~The hydrostatic load due to the water inside the UHS basin.~~

This load is only applicable to UHS/RSW Pump House. The hydrostatic load due to water inside the UHS basin is conservatively calculated considering the maximum water height of 71 ft above the top of the UHS basin basemat. The maximum hydrostatic pressure is 4.43 ksf at the top of UHS basin basemat elevation.

3H.6.4.3.3.3 Lateral Soil Pressures Including the Effects of SSE (H')

~~This is the total lateral soil pressure, including the dynamic effect of SSE.~~

The calculated lateral soil pressures including the effects of SSE are presented in figures as indicated:

- Lateral soil pressures for design of UHS/RSW Pump House: Figures 3H.6-41 through 3H.6-43.
- Lateral Soil pressures for design of RSW Piping Tunnels: Figures 3H.6-44.
- Lateral soil pressures for stability evaluation of UHS/RSW Pump House: Figures 3H.6-45 through 3H.6-50.

3H.6.4.3.3.4 Extreme Environmental Flood (FL)

~~See Subsection 3H.6.4.2.3.~~

The design basis flood level is 40.0 ft MSL, in accordance with Subsections 2.4S.2.2 and 3H.6.4.2.3. The flood water unit weight ~~is conservatively considered as 80 pcf to account for minor debris in the flood water. The maximum hydrodynamic force due to design basis flood is 44 psf. The maximum pressure on the UHS/RSW Pump House due to the design basis flood is 0.524 ksf at grade level (34.0 ft MSL).~~, considering maximum sediment concentration, is 63.85 pcf per Section 2.4S.4.2.2.4.3. The design requirements for this flood, including hydrostatic, hydrodynamic, and floating debris loading, are included in Section 3.4.2.

3H.6.4.3.4.3 Reinforced Concrete Load Combinations

$$U = 1.4D + \cancel{1.7}1.4F + 1.7L + 1.7H + 1.7R_o$$

$$U = 1.4D + \cancel{1.7}1.4F + 1.7L + 1.7H + 1.7W + 1.7R_o$$

$$U = D + F + L + H + Ta + E'$$

$$U = D + F + L + H + T_o + R_o + W_t$$

$$U = D + F + L_o + H' + T_o + R_o + E'$$

$$U = 1.05D + 1.05F + 1.3L + 1.3H + 1.2T_o + 1.3R_o$$

$$U = 1.05D + 1.05F + 1.3L + 1.3H + 1.3W + 1.2T_o + 1.3R_o$$

$$U = D + F + L + H + T_o + R_o + FL$$

$$U = D + F + L + H + T_o + R_o + S_E$$

~~For the UHS basin, the required strength defined by the above load combinations are multiplied by the following Environmental Durability Factors defined in ACI 350:~~

- ~~Flexural strength.... 1.3~~
- ~~Axial tension (including hoop tension) 1.65~~
- ~~Excess shear strength carried by shear reinforcement... 1.3~~

3H.6.4.3.4.4 ACI 350 Reinforced Concrete Load Combinations for UHS Basin Design

ACI 350 requirements are applicable to portions of environmental engineering concrete structures where durability, liquid-tightness, or similar serviceability are considerations. Therefore, the ACI 350 requirements and load combinations listed in this section are applicable only to the UHS basemat and basin walls below the maximum water level elevation.

Per ACI 350, although fluid densities and heights are usually well known, the load factor for fluid loads should be taken as 1.7 as part of the concept of environmental durability and long-term serviceability. ACI 350 states that the required strength from ACI 350 load combinations shall be multiplied by the following environment durability factors:

- Flexural strength... .1.3
- Axial tension (including hoop tension)... ....1.65
- Excess shear strength carried by shear reinforcement... .1.3

In addition to the reinforced concrete load combinations listed in Section 3H.6.4.3.4.3, the UHS basemat and basin walls below the maximum water level elevation are also designed for the load combinations listed below with ACI 350 durability factors applied. Except durability factors need not be applied for the hydrostatic leak-tightness testing condition, which is a temporary loading where environmental durability and long term serviceability are not required. The hydrostatic leak-tightness testing load combination uses a load factor of 1.4 on the fluid load because it is not a long-term serviceability condition that requires a load factor of 1.7. Per ACI 350, durability factors need not be applied to load combinations that include earthquake loads. As stated in Section 3H.6.4.3.1.5, the design thermal loads used in ACI 350 load combinations should be based on most probable temperature values, rather than the upper bound temperature values.

$U = 1.4D + 1.7F + 1.7L + 1.7H$

$U = 1.4D + 1.7F + 1.7L + 1.7H + 1.7W$

$U = 1.4D + 1.4F + 1.7W$ (Hydrostatic leak-tightness testing)

$U = 1.4D + 1.7F + 1.4\,T_o + 1.3H$

RAI 03.08.04-28

QUESTION:

Follow-up to Question 03.08.04-19

In its response to Question 03.08.04-19 (Letter No. U7-C-STP-NRC-100093 dated April 29, 2010), the applicant provided some information about the foundation waterproofing material. However, some of the information provided needs further clarification. In order for the staff to conclude that the foundation waterproofing used is adequate for providing waterproofing, and will not compromise sliding stability of structures, the applicant is requested to provide the following additional information:

1. The applicant stated in its response that a two-coat elastomeric spray-on membrane will be used for waterproofing, and the physical properties of the membrane have been specifically designed to cope with the rigorous requirements of below grade conditions. However, the applicant did not provide any information regarding the meaning of "rigorous requirements of below grade conditions," and how the physical properties of the membrane meet these requirements. The applicant is requested to describe the rigor of the requirements of the below grade conditions, and how the physical properties of the membrane meet these requirements. Please also include in the in the FSAR description and thickness of the material used for the waterproof membrane.

2. The applicant stated in the response that the waterproofing membrane will be 120 mils thick, and a qualification program, which will include testing, will be developed to demonstrate that the selected material will meet the waterproofing requirements. However, the applicant did not provide any information about what the waterproofing requirements are, and the criteria to be used for the testing. Therefore, the applicant is requested to describe these waterproofing requirements to be tested including how these requirements are established, and how they will be tested to demonstrate that the selected membrane is adequate to meet the waterproofing requirements considering long term behavior of the membrane. The applicant is also requested to update the FSAR as appropriate.

3. In response to the staff's question regarding the coefficient of friction for the waterproofing membrane, the applicant has proposed an ITAAC that states that "Type testing will be performed to determine the minimum coefficient of friction of the type of material used in the mudmat-waterproofing-mudmat interface beneath the basemats of the Category I structures." It is not clear from the description if the thickness of the specimen tested will be the same as that used for the membrane. The applicant is requested to clarify this and revise the ITAAC. Also, the acceptance criteria for the ITAAC states that "A report exists and documents that the waterproof system (mudmat-waterproofing-mudmat) has a coefficient of friction to support the analysis against sliding." The applicant stated in the response that the minimum coefficient of friction needed for maintaining the minimum factor of safety against sliding for the

Reactor Building (RB) and the Control Building (CB) is 0.47. In its response, the applicant also presented in Table RAI 03.08.04-19a the minimum coefficient of friction provided at the structural concrete fill and waterproofing membrane interface as 0.6. The applicant is requested to clarify which value of coefficient of friction will be used for the acceptance criteria of the ITAAC, and include in the FSAR the minimum coefficient of friction provided at the waterproofing membrane and structural concrete fill interface. Please also revise the ITAAC acceptance criteria accordingly.

4. The applicant stated in its response (Table RAI 03.08.04-19a) that the coefficient of friction provided at the interface of the bottom of the gravel layer and soil to be the smaller of 0.6 and shear capacity of the soil. Elsewhere in the response, the applicant stated that the soil capacity exceeds the value of 0.47 needed for maintaining minimum factor of safety against sliding of RB and CB. The applicant is requested to clarify the minimum coefficient of friction available at the bottom of gravel and soil interface based on site-specific soil properties and explain how it is determined.

RESPONSE:

1. As shown in COLA Part 2, Tier 2, Table 2.5S4-8, the existing soil has a pH in the range of 7.7 to 9.3, has a chloride content of up to 1230 mg/kg (ppm) and a sulfate content of up to 622 mg/kg (ppm), with the high chloride and sulfate contents occurring in Stratum A. As part of meeting the requirements for below grade soil conditions, the selected membrane will be tested for resistance to the high pH, chloride and sulfate contents. The description and thickness of the membrane material was given in the revised response to RAI 03.08.04-19 (see STPNOC letter U7-C-STP-NRC-100093, dated April 29, 2010). The COLA markup for the description and thickness of the membrane material, as well as the requirement to test for resistance to the high pH, chloride and sulfate contents, is included at the end of this response.

2. The membrane will be tested in accordance with ASTM D5385, "Standard Test Method for Hydrostatic Pressure Resistance of Waterproofing Membranes", which requires that the membrane be subjected to a pressure of 100 psi. The acceptance criterion will be that the sample is able to resist the expected hydrostatic pressure. Based on a maximum water head of less than 90 ft (based on the depth of the Reactor Building foundation), the design hydrostatic pressure is less than 40 psi. Accelerated aging test results will be used to show that there is negligible change in the material properties or composition for at least the 60 year life of the plant. The margin provided by the test pressure of 100 psi (the design pressure is 40 psi) along with the results from accelerated age testing will ensure that the waterproofing will sufficiently resist the design hydrostatic pressure over its intended lifetime. This is included in the COLA markup included at the end of this response.

3. The thickness of the membrane to be tested will be the same as the actual nominal thickness used for the membrane. The ITAAC in COLA Part 9, Table 3.0-13 is revised

to state this as shown at the end of this response. The acceptance criterion for the minimum coefficient of friction is 0.6 and the revised ITAAC states this. The COLA markup included at the end of this response indicates that the minimum coefficient of friction provided at the waterproofing membrane and structural concrete fill interface is 0.6.

4. The bottom of gravel and soil interface is governed by the friction forces that develop under the Reactor Building and Control Building resulting from the properties of the existing materials under the buildings. The interface between the bottom of gravel and sandy soil for the Control Building will have a coefficient of friction of 0.70 for static loading based on the tangent of the friction angle (φ) as provided by COLA Part 2, Tier 2, Table 2.5S.4-37B for the Reactor Building and COLA Part 2, Tier 2, Table 2.5S.4-38B for the Control Building, but is reduced to two-thirds the value in order to compensate for repeated cyclic (seismic) loading, bringing the resultant coefficient of friction to 0.47.

   The coefficient of friction needed to maintain the minimum factor of safety was reported as 0.47 in Revision 1 of the response to RAI 03.08.04-19 (STPNOC letter U7-C-STP-NRC-100093, dated April 29, 2010). The evaluations were based on the available coefficient of friction and showed sufficient margin in the required passive pressure to be developed.

   Part of the Reactor Building will be constructed over clay, rather than sandy soil. The resistance to sliding for these locations is based on cohesion of the clay (3.4 ksf) as provided in COLA Part 2, Tier 2, Table 2.5S.4-37B. The evaluations for this case similarly showed sufficient margin in the required passive pressure to be developed.

COLA will be revised as shown below as a result of this response and will completely supersede COLA revisions provided in RAI 03.08.04-19 (see STPNOC letter U7-C-STP-NRC-100093, dated April 29, 2010). The revisions to the COLA markup provided in RAI 03.08.04-19 are shown by revision bars in the margin.

1. COLA Part 2, Tier 2, Section 3.8.6.1 will be revised as follows:

3.8.6.1 Foundation Waterproofing

The following standard supplement addresses COL License Information Item 3.23.

Foundation waterproofing is done by placing a ~~chemical agent on the exposed concrete surface of the mudmat~~ waterproofing membrane near the top elevation of the concrete fill. The ~~concrete foundation is poured directly onto the concrete mudmat~~ remainder of the concrete fill is then poured on top of the waterproofing material. A waterproof membrane that could degrade the ability of the foundation to transfer loads is not used.

The material used for the waterproof membrane will be a two-coat color-coded Methyl Methacrylate (MMA) resin, which is an elastomeric "spray-on" membrane. The total thickness of the waterproofing membrane will be a nominal 120 mils. The selected membrane will be tested for resistance to the high pH, chloride and sulfate contents shown in Table 2.5S4-8.

The membrane will be tested in accordance with ASTM D5385, Standard Test Method for Hydrostatic Pressure Resistance of Waterproofing Membranes, which requires that the membrane be subjected to a pressure of 100 psi. The acceptance criterion is that the sample is able to resist the expected hydrostatic pressure. Accelerated aging test results will be used to show that there is negligible change in the material properties or composition for at least the 60 year life of the plant. The margin provided by the test pressure of 100 psi (the design pressure is less than 40 psi) along with the results from accelerated age testing will ensure that the waterproofing will sufficiently resist the design hydrostatic pressure over its intended lifetime.

The coefficient of friction of the waterproofing material will be determined with a qualification program prior to procurement of the material. The qualification program will be developed to demonstrate that the selected material will meet the waterproofing and friction requirements. The qualification program will include testing to demonstrate that the waterproofing requirements and the coefficient of friction required to transfer seismic loads have been met. Testing methods will simulate field conditions to demonstrate that the minimum required coefficient of friction of 0.60 is achieved by the structural concrete fill - waterproof membrane structural interface.

The test program will be based on the test methods contained in ASTM D1894. The tests will be performed with the expected range of normal compressive stresses. The coefficient of friction, as defined in ASTM D1894 is the ratio of the force required to move one surface over another to the total force applied normal to those surfaces. The test fixture assembly will be designed to obtain a series of shear / lateral forces and the corresponding applied normal compressive loads. The test data will be generally represented by a best fit straight line whose slope is the coefficient of friction.

2. COLA Part 9 will be revised to add the following site-specific ITAAC.

3.0 Site-Specific ITAAC

The reference ABWR DCD Tier 1, Chapter 4.0, "Interface Requirements," identifies significant design provisions for interface between systems within the scope of the ABWR standard design and other systems that are wholly or partially outside the scope of the ABWR standard design. The interface requirements define the attributes and performance characteristics that the out-of-scope (site-specific) portion of the plant must have in order to support the certified ABWR design.

The STP 3 & 4 site-specific systems that require ITAAC because they have a safety-related, safety-significant, or risk significant function are listed below:

- Ultimate Heat Sink (UHS)
- Offsite Power System
- Makeup Water Preparation (MWP) System
- Reactor Service Water (RSW) System
- Communication System (See Section 4.0 - Emergency Planning ITAAC)
- Site Security (See Section 5.0 - Physical Security ITAAC)
- Circulating Water (CW) System
- Backfill under Category 1 Structures
- Breathing Air (BA) System
- Waterproofing Membrane

Table 3.0-13
Waterproofing Membrane

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
1. The friction coefficient to resist sliding beneath the basemat of Category I structures is at least 0.60.	1. Type testing will be performed on a membrane of the material and thickness specified for the waterproof system to determine the minimum coefficient of friction of the type of material used in the mudmat-waterproofing-mudmat interface beneath the basemats of the Category I structures.	1. A report exists and documents that the waterproof system (mudmat-waterproofing-mudmat interface) has a coefficient of friction of at least 0.60 to support the analysis against sliding.

RAI 03.08.04-29

QUESTION:

Follow-up to Question 03.08.04-22

In its response to Question 03.08.04-22 (letter no. U7-C-STP-NRC-100036 dated February 10, 2010), the applicant provided marked-up FSAR pages with information about loadings to be used for design of site-specific seismic category I structures. To assist staff in understanding the information provided, the applicant is requested to provide the following additional information/clarifications:

1. FSAR mark-up for Section 3H.6.4.3.1.5 includes a statement "This thermal condition is applicable only for the basin basemat and basin walls below the 71 ft maximum water level with ACI 350-01 durability factors" for thermal conditions described in sub item (3) and sub item (6). Please clarify why the statement is applicable for only the above two thermal conditions, and not for all 6 thermal conditions.

2. FSAR mark-up for Section 3H.6.4.3.4.3 included in the response provides loading combinations to be used for site-specific seismic category I structures. Please explain the following loading combinations:

   - $D + F + L + H + T_a + E'$ – Provide justification for using only lateral soil pressure H, and not H', which includes seismic effects.

   - $D + F + L_0 + H' + T_0 + R_0 + E'$ – Provide justification for using L_0, which is only 25% of design live load, and not L, the full design live load.

RESPONSE:

1. According to ACI 350-01, Section 9.2.8, the required strength *U* shall be multiplied by the following environmental durability factors (S) in portions of an environmental engineering concrete structure where durability, liquid-tightness, or similar serviceability are considerations. Section 1.1.1.1 defines environmental engineering concrete structures as concrete structures intended for conveying, storing, or treating water, wastewater, or other non-hazardous liquids. As a result of the requirement in Section 9.2.8, durability factors were applied to the areas of the basin walls, foundation mat, and columns that are directly in contact with the water (at and below the maximum water level height of 71 ft.).

   In addition, according to ACI 350-01, Section 9.2.8, environmental durability factors are used to reduce stresses and the resultant crack widths under service loads. Consequently, the durability factors were not applied to the winter and summer thermal conditions which have accident basin water temperatures (see sub items 1 and 4 of the response to RAI 03.08.04-22, submitted with U7-C-STP-NRC-100036 dated February 10, 2010).

Finally, Section 9.2.7 of ACI 350-01 states that estimations of temperature change shall be based on a realistic assessment of such effects occurring in service. The term "realistic assessment" is used to indicate that the most probable values rather than the upper and lower bound values of the variables should be used. As a result of this statement, the durability factors are not applied to the winter and summer thermal conditions which have minimum basin water temperatures (see sub items 2 and 5 of the response to RAI 03.08.04-22).

2. The lateral soil pressure H' is included in the Finite Element Analysis (FEA) SAP 2000 model. The load combination in Section 3H.6.4.3.4.3 will be revised to reflect the loads used in the analysis as shown in the attached COLA mark up.

   In response to RAI 03.08.04-20 submitted with U7-C-STP-NRC-100035, dated February 4, 2010, the COLA mark-up for Section 3H.6.4.3.4.3 was revised to reflect that the full design live load (L) was used in the design of local elements such as beams and slabs. It is noted explicitly in the mark-up that the expected live load present during normal plant operation, defined as 25% of the live load, is only used for the global effects of the seismic live load.

The COLA mark-up for Section 3H.6.4.3.4.3 provided in the response to RAI 03.08.04-22 will be revised as shown:

3H.6.4.3.4.3 Reinforced Concrete Load Combinations

$$U = D + F + L + \boxed{\cancel{H}H'} + T_a + E'$$

RAI 03.08.04-31

QUESTION:

Follow-up to Question 03.08.04-25

The staff reviewed the applicant's response to Question 03.08.04-25 (letter U7-C-STP-NRC-100108, dated May 13, 2010). In order for the staff to conclude that the interface between seismic category I buildings and tunnels will not result in any unacceptable interaction, the applicant is requested to provide the following additional information:

1. The applicant stated in its response that the separation gap between the Reactor Service Water (RSW) Piping Tunnels and the RSW Pump House and the Control Building (CB), as well as between the Diesel Generator Fuel Oil Storage Vaults (DGFOSV) and the Diesel Generator Fuel Oil Tunnels (DGFOT), will be at least 50% larger than the absolute sum of the calculated displacements due to seismic movements and long term settlement. The material used as flexible filler will be able to be compressed to approximately 1/3 of its thickness without subjecting the building to more than a negligible force. However, the applicant provided vendor test result where 7 psi compressive stress was observed when 5 inch joint was compressed to 50% movement. This does not provide any estimate of how much compressive stress may be developed when the material is compressed to 1/3 thickness of the material. Therefore, the applicant is requested to justify that no significant stress will be imparted to the building when the joint is compressed to 1/3 thickness.

2. The DGFOT is connected to the DGFOSV at one end. It is not clear from the response where the DGFOT is connected at the other end, and what are the anticipated movements at that connection. Please include this information in Table 3H.6-15.

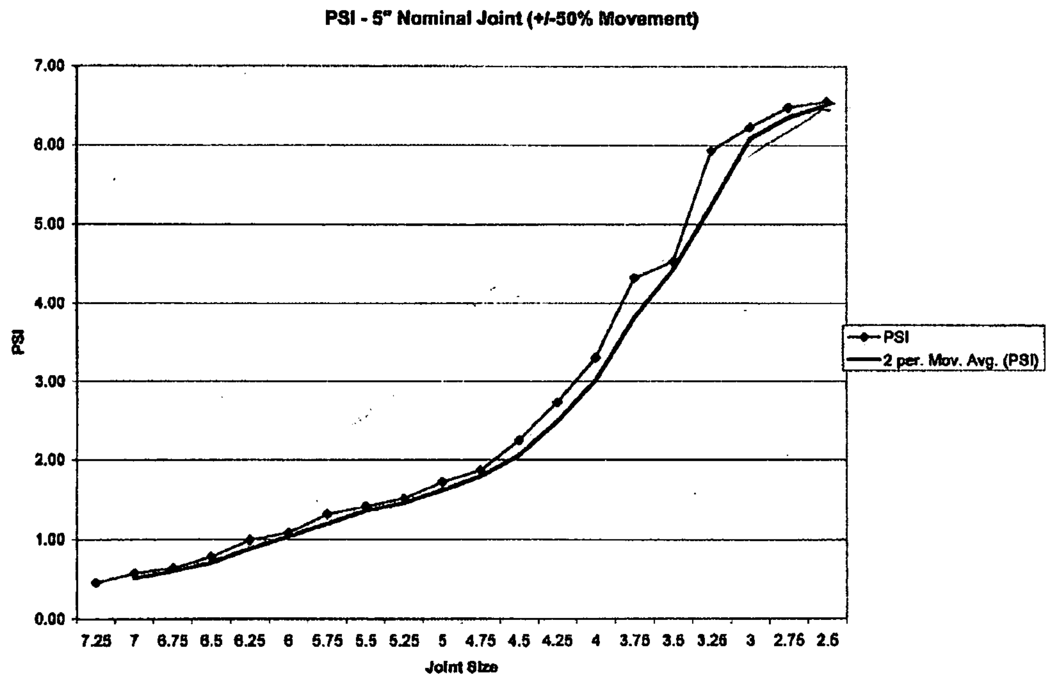
3. Please provide an ITAAC with key parameters for as-built verification of the connections, or provide justification for not doing so.

RESPONSE:

The following provides the response to parts 1 and 3 of this RAI. The response to part 2 of this RAI will be provided in a supplemental response by November 15, 2010.

1. The actual material for seals has not been selected, nor has it been tested for the compressive stress applied when it is compressed to 1/3 of the original joint size. However, based on the following graph representing typical vendor data, which shows the compressive stress of the joint filler when the joint is expanded and contracted by 50%, the relationship between the joint size and the compressive stress appears to be approximately linear throughout the compression zone. The graph shows that the compressive stress is approximately 1.6 psi when installed in a 5" nominal joint. This

stress decreases to 0.5 psi when the joint expands to approximately 150% of the original size (7.25"), and increases to 6.5 psi when the joint contracts to 50% of the original size (2.5"). Therefore, there is sufficient confidence that the compressive stress will be less than 25 psi when compressed to 1/3 of the original joint size.



The COLA will be updated, as shown in Enclosure 1, to require the maximum compressive stress of the material to be less than 25 psi when subjected to the maximum static and dynamic differential displacements of the joints.

Based on ACI 349-97 Section 10.15, the bearing capacity of 4000 psi concrete is 2380 psi, which is significantly higher than the maximum pressure of 25 psi that may be applied at the seismic joint. Therefore local effects of this load are considered negligible.

The structures experiencing the load from the seal material are either loaded in-plane (e.g. Reactor Service Water (RSW) Piping Tunnels) or out-of-plane (e.g. RSW Pump House walls). For structures loaded in-plane, the axial capacity of the concrete section is over 1800 psi based on ACI 349-97 Section 10.3.5.2. This is significantly higher than the 25 psi load that may be exerted by the filler material at the joint locations. Concrete walls loaded out-of-plane by the seal material have also been designed for a minimum 15 psi soil pressure load during seismic events. Since the area where the filler material will be placed is very small in comparison to the area of the wall loaded by static and dynamic soil pressure, the pressure exerted by the filler material is insignificant compared to the total applied soil pressure. Therefore global effects on the walls are considered negligible.

3. In accordance with the response to question 1, the COLA will be updated to state that maximum compressive stress of the material will be less than 25 psi when subjected to the maximum static and dynamic differential displacements of the joints. Because an appropriate material will be selected to meet this COLA requirement, there will be no significant stress imparted to the building when the joint is compressed to 1/3 thickness. Therefore, an additional ITAAC commitment is not required.

The COLA revision submitted with this response for Section 3H.6.8 completely supersedes the COLA revision for Section 3H.6.8 submitted with response to RAI 03.08.04-25, Revision 2 (submitted with STPNOC letter U7-C-STP-NRC-100108, dated May 13, 2010).

RAI 03.08.04-31
Enclosure 1

New COLA Part 2, Tier 2, Section 3H.6.8

Add the following new subsection 3H.6.8 and revise the subsequent subsection numbers.

3H.6.8 Seismic Gaps at the Interface of Site-Specific Seismic Category I Structures and the Adjoining Structures

The joints (i.e. separation gaps) at the interface of site-specific seismic category I structures (Reactor Service Water Tunnels and Diesel Generator Fuel Oil Storage Vaults) with the adjoining structures (Control Buildings, Reactor Service Water Pump Houses, and Diesel Generator Fuel Oil Tunnels) are designed to accommodate the expected movements without transmitting significant forces. These separation gaps are sized at least 50% larger than the absolute sum of the maximum calculated displacements due to seismic movements and long term settlement. The joint material used as flexible filler will be polyurethane foam impregnated with a waterproofing sealing compound, or a similar material, capable of being compressed to 1/3 of its thickness without subjecting the structures to more than 25 psi. The walls of the Reactor Service Water Pump House and the Diesel Generator Fuel Oil Storage Vaults have been evaluated and found to be adequate for this out-of-plane load.
Table 3H.6.15 provides a summary of the required and provided gaps at the interface of site-specific seismic category I structures with adjoining structures.

~~3H.6.8~~3H.6.9 References

RAI 03.08.04-32

QUESTION:

Follow-up to Question 03.08.04-27

The applicant stated in its response (letter U7-C-STP-NRC-100036, dated February 10, 2010) to Question 03.08.04-27 regarding COL License Information Item 3.25 that the details of the Structural Integrity Test (SIT) and the instrumentation required for the test will be provided in the ASME Construction Specification. The applicant referred to RG 1.206, Section CIII.4.3, situation 4 for resolving the COL information item six months before performance of the test. According to RG 1.206, Section CIII.4.3, the applicant should justify why the item is not resolved before the issuance of license. However, the applicant did not provide any justification. Therefore, the applicant is requested to provide a detailed justification for why any part or all of the information pertaining to the COL information item cannot be provided at this time and clearly addressing all parts of COL license information item. Also, the applicant is requested to identify in Chapter 1 of the FSAR if the COL information item cannot be resolved completely before the COL is issued. The staff needs this information to conclude that deferral of the COL information item meets the guidance provided in RG 1.206.

RESPONSE:

The response to this RAI completely supersedes the responses to RAI 03.08.04-6 (provided in letter U7-C-STP-NRC-090136 dated September 15, 2009) and RAI 03.08.04-27 (provided in letter U7-C-STP-NRC-100036 dated February 10, 2010).

Details of the Test and Instrument Plan for the Structural Integrity Test (SIT) are provided below. The Unit 3 Reinforced Concrete Containment Vessel (RCCV) is classified as a prototype containment. Therefore, the test and instrument plan for the Unit 3 SIT has been developed to conform to the requirements for prototype containments as delineated in Article CC-6000 of ASME Section III, Division 2. The test and instrument plan for the Unit 4 SIT will conform to the requirements for non-prototype containments as delineated in Article CC-6000 of ASME Section III, Division 2.

The following is a summary of SIT requirements for Units 3&4 based on Article CC-6000 of ASME Section III, Division 2. These will be included in the ASME Construction Specification for the Containment.

I. Details of the Test:

The containment shall be subjected to integrity tests that include both an overall internal pressure test and a differential pressure test. The overall SIT will be performed at a test pressure of at least 1.15 times the containment design pressure in both the drywell and suppression chamber simultaneously. The differential pressure test will be performed at a test pressure of at least 1.0 times the maximum design differential pressure. The test pressure will be held for at least 1 hour. Predictions of strains (Unit 3 only) and displacements will be made prior to the start of the test.

During the SIT, the suppression chamber and spent fuel pool will be filled with water to the normal operational water level. Atmospheric air will be used as the testing medium for both the overall and the differential pressure test. The Designer or his designee will perform a pretest visual examination of the accessible portions of the RCCV prior to the SIT in accordance with CC-6210 of ASME Section III, Division 2. The Designer or his designee will witness the SIT and will monitor displacement measurements.

1. <u>Test Description & Objectives</u>

   a. The SIT will test the RCCV for structural performance acceptability as a prerequisite for Code Acceptance and stamping. The test will be conducted in accordance with the 2001 Edition, including 2003 addenda, of the ASME Boiler & Pressure Vessel Code, Section III, Division 2, Article CC-6000 (hereinafter referred to as the ASME Code).

   b. The SIT is performed at a test pressure of at least 1.15 times the containment design pressure of 45 psig (1.15x45=51.75 psig) to demonstrate the quality of construction and to verify the acceptable performance of new design features. The structural response of the system under the required maximum test pressure - measured in terms of displacements, strain (Unit 3 only) and cracking - shall be recorded and the data shall be presented in a final report.

   c. Evaluation of SIT results will be conducted in accordance with Section CC-6400 of the ASME Code using the acceptance criteria given in Section CC-6410.

   d. The SIT shall be performed using atmospheric air.

2. <u>Test Parameters:</u>

   a. Loading

      i. Pressurization/depressurization of the RCCV

      The SIT will subject the RCCV to a pressurization/depressurization sequence during which the internal pressure is increased from atmospheric pressure to the test pressure at which point pressure inside the RCCV will be held at maximum test pressure for at least 1 hour. Afterwards, the internal pressure is decreased from the maximum test pressure to atmospheric pressure. A detailed description of the test pressurization sequence is provided in Section I.2.a.iii below.

ii. Differential pressurization/depressurization of drywell and suppression chamber

The SIT will subject the drywell of the RCCV to a differential pressurization/depressurization sequence while the suppression chamber is at the atmospheric pressure. For this test, the internal pressure of the drywell is set to 25 psig and held at this level for at least 1 hour.

iii. Pressurization Sequence

The pressurization/depressurization rate during the test shall not exceed 20% of the maximum test pressure per hour, or 10.35 psig per hour. The pressurization and depressurization shall be performed using a minimum of 5 pressure steps. At the end of each step, the pressure shall be held for a minimum of 1 hour to collect a full set of strains (Unit 3 only), displacements, and temperatures. Once the full SIT test pressure is obtained, the pressure shall be held for a minimum of 2 hours to perform crack mapping in addition to collecting a full set of strains (Unit 3 only), displacements, and temperatures. The same process shall be used during the depressurization phase of the test.

b. Response

i. Displacement

Displacement measurements shall be taken at the following locations:

1 Radial displacements in the drywell: top of the upper drywell, mid-height of the upper drywell, and above the diaphragm floor. Radial displacements in the suppression chamber (SC): top of the SC, mid-height of the SC, and above the basemat. Measurements shall be made at a minimum of four approximately equally spaced azimuths and should be perpendicular to the containment centerline.

2 Radial displacements of the containment wall adjacent to the largest opening, at a minimum of 12 points, four equally spaced on each of three concentric circles. The diameter for the inner circle shall be large enough to permit measurements to be made on the concrete rather than on the steel sleeve; the middle approximately 1.75 times the diameter of the opening; and the outer approximately 2.5

times the diameter of the opening. The change in the diameter of the opening shall be measured on the horizontal and vertical axes.

3 Vertical displacement of the RCCV walls at the top of the drywell relative to the basemat–wall junction, measured at a minimum of four approximately equally spaced azimuths.

4 Vertical displacement of the drywell top slab relative to the basemat near the reactor shield wall, and vertical displacement of the drywell top slab relative to the basemat at two other approximately equally spaced locations between the reactor shield wall and the primary vertical wall of the RCCV on a common azimuth.

ii. Strain (Unit 3 only)

Per requirements of Section CC-6370 of ASME code, the Unit 3 prototype containment shall be instrumented to measure strain. At a minimum, strain measuring instrumentation will be located at two azimuths, 90 degrees apart, to demonstrate the structural behavior of the following areas of the RCCV:

- the intersection of the shell and the basemat.
- near mid-height on the suppression chamber.
- near mid-height on the upper drywell.
- the vicinity of the lower drywell access tunnel at azimuth 180 deg.
- the intersection of the shell and the top slab.
- the intersection of the shell and the diaphragm floor.
- the intersection of the top slab and the drywell head.

iii. Temperature

Ambient temperature shall be measured inside and outside the RCCV. In addition, per requirements of Section CC-6380 of ASME code, for the Unit 3 prototype containment, temperatures shall be measured at all strain gage locations to establish representative temperatures for strain measurements. Temperature measurements shall be used to correct measured strain values for thermal effects.

iv. Crack mapping

Per requirements of Section CC-6350 of ASME code, concrete surface cracks shall be mapped. The patterns of cracks that exceed 0.01 in (0.25 mm) in width and 6 in. (152 mm) in length shall be mapped at specified locations before the test, at maximum pressure, and after the test. Locations shall be as specified by the Designer and shall include areas where high surface tensile strain is predicted. At each location, an area of at least 40 sq ft (3.7 m^2) shall be mapped.

v. Post-test examination

A post-test examination will be made within one (1) week of depressurization. Details of the post-test examination will be the same as those of the pretest examination required by CC-6210 of ASME Section III, Division 2.

II. Instrumentation:

Instrumentation for the measurement of pressure, displacement, strain (for Unit 3), crack width and length, and temperature will be provided in accordance with CC-6220 of ASME Section III, Division 2. Output of all instruments will be recorded prior to start of testing and any erratic readings corrected, if possible, or noted. All malfunctioning instrumentation will be reported to and evaluated by the Designer before proceeding with testing. Instruments that become erratic or inoperative during testing will be reported to the Designer before proceeding with testing.

Displacement, strain (for Unit 3), and temperature measurements will be made in accordance with CC-6300 of ASME Section III, Division 2. Test data will be collected in accordance with CC-6340 of ASME Section III, Division 2. For the prototype Unit 3 Containment, strains and associated temperatures will be measured for a minimum period of 24 hours prior to the SIT to evaluate the strain variations resulting from temperature change.

1. <u>Equipment Description</u>

a. Pressurization system

(a) The pressurization system shall be capable of attaining and holding the maximum test pressure of 51.75 psig during the pressurization/ depressurization of the RCCV and a test pressure of 25 psig during the differential pressurization/depressurization of the drywell and suppression chamber.

(b) Equipment inside the RCCV that will be subject to pressure from the SIT sequence shall be prepared for the test appropriately, including potential for water vapor condensation.

b. Data acquisition system specifications

(a) Data loggers will be used to collect data from various system components including thermometers, strain gages, pressure gages, and displacement transducers. Input/output measurement and control modules, multiplexers, communication interface equipment, battery backup power supplies and signal conditioning equipment shall be supplied as necessary based upon the configuration and features of the instrumentation equipment used.

(b) The data loggers shall have appropriate non-volatile on-board memory to minimize inadvertent loss of data. Sufficient data storage capacity will be provided to store data collected from all gages during the structural integrity test without interruption.

(c) Data collected from all gages shall have a time stamp.

c. Specifications for instrumentation

(a) Sister bar strain gages
Sister bar strain gages are the preferred choice for measurement of strain in reinforcing steel.

1 Sister bar strain gages will be properly secured to the rebar cage at pre-defined locations (indicated in Section I.2.b.ii above) and embedded in the concrete during concrete placement. The end-to-end length of the bar segment used for the sister bar strain gages shall be two times the development length of the sister bar plus either 4 in. or the protected length of the sister bar, whichever is greater. The sensing components shall be foil type resistance strain gages as described below. The foil type resistance strain gages shall be installed in a full bridge, 4-arm configuration for improved stability. The gages shall be mounted at two locations around the circumference of the sister rebar at mid-length. The two locations shall be positioned at ± 180 degrees from each other. The strain gages shall be bonded to the sister bar by strain gage epoxy if directly attached to the rebar, or spot welded if previously encapsulated inside a stainless steel shim. The rebar surface at the location of the strain gage attachment shall be prepared according to the strain gage manufacturer installation requirements. A thermistor shall also be attached to the rebar, near the strain gages, to permit the differentiation of thermally induced strains from load induced strains. The strain gages and thermistor shall be protected against moisture and chemical and mechanical damage. Moisture protective material shall

be a type used for underwater applications such as silicone. A protective coating such as polysulfide shall be applied over the water proofing material to protect the strain gages against mechanical and chemical damages. A heat shrinkage protector shall be further applied over the protective coatings for further reinforcement. Each fabricated sister bar strain gage shall be tested by complete water immersion for at least 24 hrs. The sister bar element shall be supplied with an appropriate cable as defined in Section II.1.d. with an appropriate length of cable such that there are no cable splices inside the concrete. In addition, when splices are required outside the concrete, all connections shall be soldered and then protected from moisture and other contamination with a suitable cable splice sealant. The cables shall be waterproofed and sealed as an integral part of the assembly.

2 The foil type strain gages shall have following characteristics:

a. Standard Range	3000 micro strain
b. Sensitivity	1 micro strain
c. Accuracy	5% of the maximum anticipated strain or 10 microstrain, whichever is greater

(b) Displacement transducer

1 Linear variable displacement transducers (LVDTs) shall be used for both vertical and horizontal displacement measurements. Inside the suppression chamber submersible LVDTs shall be used for measurement locations that are below the water line.

2 LVDTs shall have the following minimum characteristics:

a. Travel Range	0.5 in
b. Output	4-20 mA
c. Minimum Linearity	$\pm$0.30% full scale
d. Min Repeatability	$\pm$0.015% full scale

(c) Temperature gage

1 Temperature devices shall be resistance type and shall be sealed against moisture. Thermistors used in fabrication of sister bar gages shall have diffusivity approximately that of steel.

2 Temperature sensing element shall be supplied with an appropriate cable as defined in Section II.1.d. The cables shall be waterproofed and sealed as an integral part of the assembly.

(d) Pressure gage

1 Pressure gages used in pressure testing shall be connected directly to the internal environment of the containment, and measure the differential pressure between the internal and external environments. This shall be accomplished either by using an absolute pressure gage inside and another absolute gage outside of the RCCV or by using a gage-pressure gage directly attached to the pressurizing pump outlet outside of the RCCV right after the shut-off valve. The pressure gages shall be voltage output (as compared to millivolt output type) with integrated signal conditioning electronics included. The pressure gages shall be supplied with an appropriate cable as defined in Section II.1.d. The pressure gage cables shall be waterproofed and sealed as an integral part of the assembly.

2 The pressure gages shall have the following characteristics:

a.	Range	0-200 psi
b.	Accuracy	$\pm$0.25 psi

d. Cable specifications

Instrumentation cable type and size shall be shielded 16 AWG twisted paired for all instruments. The shield shall be either braided strands of copper (or other metal), a non-braided spiral winding of copper tape (or other metal), or a layer of conducting polymer. The shield shall be applied across cable splices. In addition, the cable shall have drain wire.

III. Evaluation of Test Results:

Crack and strain (for Unit 3) measurements will be reviewed by the Designer for evaluation of the overall test results. The RCCV will be considered to have satisfied the structural integrity test if the minimum requirements specified in CC-6410 of ASME Section III, Division 2 are met. If measurements and studies by the Designer indicate that the requirements of CC-6410 are not met, remedial measures will be undertaken or a retest will be conducted in accordance with CC-6430 of ASME Section III, Division 2.

IV. Test Report:

The results of structural integrity tests will be submitted to the Designer. The report will meet the minimum requirements of CC-6530.

The COLA will be revised as provided in the enclosure to this response.

Enclosure to RAI 03.08.04-32

Revision to COLA Section 3.8.6.3

Section 3.8.6.3 of the COLA will be revised as follows:

3.8.6.3 Structural Integrity Test Result

The following standard supplement addresses COL License Information Item 3.25.

Structural Integrity Test (SIT) of the containments will be performed in accordance with Subsection 3.8.1.7.1 and ITAAC Table 2.14.1 Item #3. ~~The first~~The Unit 3 containment will be considered a prototype and its SIT performed accordingly. The details of the test and the instrumentation~~, as required for such a test, will be provided to NRC for approval~~ are provided in the following subsections.

3.8.6.3.1 Details of the Test:

The SIT will be performed at a test pressure of at least 1.15 times the containment design pressure in both the drywell and suppression chamber simultaneously. The test pressure will be held for at least 1 hour. Predictions of strains (Unit 3 only) and displacements will be made prior to the start of the test.

During the SIT, the suppression chamber and spent fuel pool will be filled with water to the normal operational water level. Atmospheric air will be used as the testing medium. The Designer or his designee will perform a pretest visual examination of the accessible portions of the Reinforced Concrete Containment Vessel (RCCV) prior to the SIT in accordance with CC-6210 of ASME Section III, Division 2. The Designer or his designee will witness the SIT and will monitor displacement measurements.

3.8.6.3.1.1 Test Description & Objectives

(1) The SIT will test the RCCV for structural performance acceptability as a prerequisite for Code Acceptance and stamping. The test will be conducted in accordance with the 2001 Edition, including 2003 addenda, of the ASME Boiler & Pressure Vessel Code, Section III, Division 2, Article CC-6000 (hereinafter referred to as the ASME Code).

(2) The SIT is performed at a test pressure of at least 1.15 times the containment design pressure of 45 psig (1.15x45=51.75 psig) (357 kPag) to demonstrate the quality of construction and to verify the acceptable performance of new design features. The structural response of the system under the required maximum test pressure - measured in terms of displacements, strain (Unit 3 only) and cracking - shall be recorded and the data shall be presented in a final report.

(3) Evaluation of SIT results will be conducted in accordance with Section CC-6400 of the ASME Code using the acceptance criteria given in Section CC-6410.

(4) The SIT shall be performed using atmospheric air.

3.8.6.3.1.2 Test Parameters:

(1) Loading

(a) Pressurization/depressurization test of the RCCV

The SIT will subject the RCCV to a pressurization/depressurization sequence during which the internal pressure is increased from atmospheric pressure to the test pressure at which point pressure inside the RCCV will be held at maximum test pressure for at least 1 hour. Afterwards, the internal pressure is decreased from the maximum test pressure to atmospheric pressure. A detailed description of the test pressurization sequence is provided in Subsection 3.8.6.3.1.2(1)(c) below.

(b) Differential pressurization/depressurization of drywell and suppression chamber

The SIT will subject the drywell of the RCCV to a differential pressurization/depressurization sequence while the suppression chamber is at the atmospheric pressure. For this test, the internal pressure of the drywell is set to 25 psig (172 kPag) and held at this level for at least 1 hour.

(c) Pressurization Sequence

The pressurization/depressurization rate during the test shall not exceed 20% of the maximum test pressure per hour, or 10.35 psig per hour. The pressurization and depressurization shall be performed using a minimum of 5 pressure steps. At the end of each step, the pressure shall be held for a minimum of 1 hour to collect a full set of strains (Unit 3 only), displacements, and temperatures. Once the full SIT test pressure is obtained, the pressure shall be held for a minimum of 2 hours to perform crack mapping in addition to collecting a full set of strains (Unit 3 only), displacements, and temperatures. The same process shall be used during the depressurization phase of the test.

(2) Response

(a) Displacement

Displacement measurements shall be taken at the following locations:

(a.1) Radial displacements in the drywell: top of the drywell, mid-height of the upper drywell, and above the diaphragm floor. Radial displacements in the suppression chamber (SC): top of the SC, mid-height of the SC, and above the basemat. Measurements shall be made at a minimum of four approximately equally spaced azimuths and should be perpendicular to the containment centerline.

(a.2) Radial displacements of the containment wall adjacent to the largest opening, at a minimum of 12 points, four equally spaced on each of three concentric circles. The diameter for the inner circle shall be large enough to permit measurements to be made on the concrete rather than on the steel sleeve; the middle approximately 1.75 times the diameter of the opening; and the outer approximately 2.5 times the diameter of the opening. The change in the diameter of the opening shall be measured on the horizontal and vertical axes.

(a.3) Vertical displacement of the RCCV walls at the top of the drywell relative to the basemat–wall junction, measured at a minimum of four approximately equally spaced azimuths.

(a.4) Vertical displacement of the drywell top slab relative to the basemat near the reactor shield wall, and vertical displacement of the drywell top slab relative to the basemat at two other approximately equally spaced locations between the reactor shield wall and the primary vertical wall of the RCCV on a common azimuth.

(b) Strain (Unit 3 Only)

Per requirements of Section CC-6370 of ASME code, the Unit 3 prototype containment shall be instrumented to measure strain. Strain measuring instrumentation will be located so as to demonstrate the structural behavior of the following areas of the RCCV, at a minimum:

(b.1) the intersection of the shell and the basemat.
(b.2) near mid-height on the suppression chamber.
(b.3) near mid-height on the upper drywell.
(b.4) the vicinity of the lower drywell access tunnel at azimuth 180 deg.
(b.5) the intersection of the shell and the top slab.
(b.6) the intersection of the shell and the diaphragm floor.
(b.7) the intersection of the top slab and the drywell head.

(c) Temperature

Ambient temperature shall be measured inside and outside the RCCV. In addition, per requirements of Section CC-6380 of ASME code, for the Unit 3 prototype containment, temperatures shall be measured at all strain gage locations to establish representative temperatures for strain measurements. Temperature measurements shall be used to correct measured strain values for thermal effects.

(d) Crack mapping

Per requirements of Section CC-6350 of ASME code, concrete surface cracks shall be mapped. The patterns of cracks that exceed 0.01 inch (0.25 mm) in width and 6 inches (152 mm) in length shall be mapped at specified locations before the test, at maximum pressure, and after the test. Locations shall be as specified by the Designer and shall include areas where high surface tensile strain is predicted. At each location, an area of at least 40 sq ft (3.7 m^2) shall be mapped.

(e) Post-test examination

A post-test examination will be made within one (1) week of depressurization. Details of the post-test examination will be the same as those of the pretest examination required by CC-6210 of ASME Section III, Division 2.

3.8.6.3.2 Instrumentation:

Instrumentation for the measurement of pressure, displacement, strain (for Unit 3), crack width and length, and temperature will be provided in accordance with CC-6220 of ASME Section III, Division 2. Output of all instruments will be recorded prior to start of testing and any erratic readings corrected, if possible, or noted. All malfunctioning instrumentation will be reported to and evaluated by the Designer before proceeding with testing. Instruments that become erratic or inoperative during testing will be reported to the Designer before proceeding with testing.

Displacement, strain (for Unit 3), and temperature measurements will be made in accordance with CC-6300 of ASME Section III, Division 2. Test data will be collected in accordance with CC-6340 of ASME Section III, Division 2. For the prototype Unit 3 Containment, strains and associated temperatures will be measured for a minimum period of 24 hours prior to the SIT to evaluate the strain variations resulting from temperature change.

3.8.6.3.2.1 Equipment Description

(1) Pressurization system

(a) The pressurization system shall be able to attain and hold the maximum test pressure of 51.75 psig (357 kPag) during the pressurization/ depressurization of the RCCV and a test pressure of 25 psig (172 kPag) during the differential pressurization/depressurization of the drywell and suppression chamber.

(b) Equipment inside the RCCV that will be subject to pressure from the SIT sequence shall be prepared for the test appropriately, including potential for water vapor condensation.

(2) Data acquisition system specifications

(a) Data loggers will be used to collect data from various system components including thermometers, strain gages, pressure gages, and displacement transducers. Input/output measurement and control modules, multiplexers, communication interface equipment, battery backup power supplies and signal conditioning equipment shall be supplied as necessary based upon the configuration and features of the instrumentation equipment used.

(b) The data loggers shall have appropriate non-volatile on-board memory to minimize inadvertent loss of data. Sufficient data storage capacity will be provided to store data collected from all gages during the structural integrity test without interruption.

(c) Data collected from all gages shall have a time stamp.

(3) Specifications for instrumentation

(a) Sister bar strain gages

Sister bar strain gages are the preferred choice for measurement of strain in reinforcing steel.

(a.1) Sister bar strain gages will be properly secured to the rebar cage at pre-defined locations (See Section 3.8.6.3.1.2(2)(b)) and embedded in the concrete during concrete placement. The end-to-end length of the bar segment used for the sister bar strain gages shall be two times the development length of the sister bar plus either 4 in. or the protected length of the sister bar, whichever is greater. The sensing components shall be foil type resistance strain gages as described below. The foil type resistance strain gages shall be installed in a full bridge, 4-arm configuration for improved stability. The gages shall be mounted at two locations around the circumference of the sister rebar at mid-length. The two locations shall be positioned at +180 degrees from each other. The strain gages shall be bonded to the sister bar by strain gage epoxy if directly attached to the rebar, or spot welded if previously encapsulated inside a stainless steel shim. The rebar surface at the location of the strain gage attachment shall be prepared according to the strain gage manufacturer installation requirements. A thermistor shall also be attached to the rebar, near the strain gages, to permit the differentiation of thermally induced strains from load induced strains. The strain gages and thermistor shall be protected against moisture and chemical and mechanical damage. Moisture protective material shall be a type used for underwater applications such as silicone. A protective coating such as polysulfide shall be applied over the water proofing material to protect the strain gages against mechanical and chemical damages. A heat shrinkage protector shall be further applied over the protective coatings for further reinforcement. Each fabricated sister bar strain gage shall be tested by complete water immersion for at least 24 hrs. The sister bar element shall be supplied with an appropriate cable as defined in Subsection 3.8.6.3.2.1(4) below with an appropriate length of cable such that there are no cable splices inside the concrete. In addition, when splices are required outside the concrete, all connections shall be soldered and then protected from moisture and other contamination with a suitable cable splice sealant. The cables shall be waterproofed and sealed as an integral part of the assembly.

(a.2) The foil type strain gages shall have following characteristics:

a. Standard Range	3000 micro strain
b. Sensitivity	1 micro strain
c. Accuracy	5% of the maximum anticipated strain or 10 microstrain, whichever is greater

(b) Displacement transducer

(b.1) Linear variable displacement transducers (LVDTs) shall be used for both vertical and horizontal displacement measurements. Inside the suppression chamber submersible LVDTs shall be used for measurement locations that are below the water line.

(b.2) LVDTs shall have the following minimum characteristics:

a.	Travel Range	0.5 in
b.	Output	4-20 mA
c.	Minimum Linearity	±0.30% full scale
d.	Min Repeatability	±0.015% full scale

(c) Temperature gage

(c.1) Temperature devices shall be resistance type and shall be sealed against moisture. Thermistors used in fabrication of sister bar gages shall have diffusivity approximately that of steel.

(c.2) Temperature sensing element shall be supplied with an appropriate cable as defined in Subsection 3.8.6.3.2.1(4) below. The cables shall be waterproofed and sealed as an integral part of the assembly.

(d) Pressure gage

(d.1) Pressure gages used in pressure testing shall be connected directly to the internal environment of the containment, and measure the differential pressure between the internal and external environments. This shall be accomplished either by using an absolute pressure gage inside and another absolute gage outside of the RCCV or by using a gauge pressure gage directly attached to the pressurizing pump outlet outside of the RCCV right after the shut-off valve. The pressure gages shall be voltage output (as compared to millivolt output type) with integrated signal conditioning electronics included. The pressure gages shall be supplied with an appropriate cable as defined in Subsection 3.8.6.3.2.1(4) above. The pressure gage cables shall be waterproofed and sealed as an integral part of the assembly.

(d.2) The pressure gages shall have the following characteristics:

a. Range	0-200 psi
b. Accuracy	±0.25 psi

(4) Cable specifications

Instrumentation cable type and size shall be shielded 16 AWG twisted paired for all instruments. The shield shall be either braided strands of copper (or other metal), a non-braided spiral winding of copper tape (or other metal), or a layer of conducting polymer. The shield shall be applied across cable splices. In addition, the cable shall have drain wire.

3.8.6.3.3 Evaluation of Test Results:

Crack and Unit 3 strain measurements will be reviewed by the Designer for evaluation of the overall test results. The primary containment will be considered to have satisfied the structural integrity test if the minimum requirements specified in CC-6410 of ASME Section III, Division 2 are met. If measurements and studies performed by the RCCV design organization indicate that the requirements of CC-6410 are not met, remedial measures will be undertaken or a retest will be conducted in accordance with CC-6430 of ASME Section III, Division 2.

3.8.6.3.4 Test Report:

The results of structural integrity tests will be submitted to the Designer. The report will meet the minimum requirements of CC-6530.

RAI 03.08.04-33

QUESTION:

1. In FSAR Section 3.8, page 3.8-1, the applicant references the departure STD DEP 1.8-1, "Tier 2* Codes, Standards, and Regulatory Guide Edition Changes." One of the changes included in this departure updates Tier 2 to refer to the 1997 edition of ACI 349 in place of the 1980 edition of the same building code for concrete structures. In the ABWR design certification (NUREG-1503, page 3-53), the staff had evaluated only the use of 1980 edition of ACI 349. Therefore, the applicant is requested to provide a detailed comparison of the differences between these two editions of the code as they apply to the ABWR standard design, and provide justifications for any differences in order for the staff to evaluate the acceptability of the 1997 edition of ACI 349.

2. FSAR Section 3H.6.4.1 references ANSI/AISC N690 specification for design, fabrication, and erection of site-specific seismic category I steel structures. The applicant did not specify in this section which version of the specification is used. It appears that the applicant uses the 1984 edition of the specification referenced in ABWR DCD Table 1.8-21, which the applicant incorporated by reference. However, according to SRP acceptance criteria 3.8.4.II.5, ANSI/AISC N690-1994 including Supplement 2 (2004) has been accepted by the staff for design, fabrication, and erection of safety-related steel structures. According to the guidance provided in RG 1.206, Section C.I.1.9.2, the applicant should use the current SRP for structures outside the scope of the ABWR DCD, or provide justification for not doing so. Therefore, the applicant is requested to provide a detailed comparison of the differences between the 1984 (or whatever edition is used by the applicant) and the 1994 editions of the specification as they apply to the site-specific seismic category I structures at STP site. Also, provide the justification(s) for any differences in order for the staff to evaluate the acceptability of the 1984 edition of the specification.

3. Furthermore, the staff observed that Table 1.8-21 in FSAR Tier 2, Section 1.8, references ASME Code, Section III, Division 2, Edition 2001 with 2003 addenda, and identifies certain limitations. The ABWR DCD specifies the use of ASME code version 1989. In the ABWR FSER, p. 3-49, the NRC has accepted the 1989 Edition of the ASME Code, Section III, Division 2. Therefore, the applicant is requested to provide a detailed comparison of the differences between these two editions of the code as they apply to the design and analysis of safety-related ABWR standard plant structures, and provide justification(s) for any differences in order for the staff to evaluate the acceptability of the ASME Code, Section III, Division 2 Edition 2001 with 2003 addenda. The applicant is also requested to explain how use of the Edition of the ASME Code proposed by the applicant meets the provisions of NCA-1140, "Use of Code Editions, Addenda, and Cases."

The staff needs the above information to conclude that the applicant used acceptable codes and standards for all seismic category I structures, and any deviations are appropriately addressed.

RESPONSE:

1. STD DEP 1.8-1, "Tier 2* Codes, Standards, and Regulatory Guide Edition Changes," includes several changes. As noted in the RAI Question, one of the changes included in this departure updates Tier 2 Table 1.8-21 to refer to the 1997 edition of ACI 349 in place of the 1980 edition. In addition, Table 1.8-20 changes the commitment for Regulatory Guide (RG) 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments)", from Revision 1 to Revision 2. Revision 2 of Regulatory Guide 1.142 endorses the 1997 edition of ACI 349.

   The use of ACI 349-97 in lieu of ACI 349-80 is consistent with Regulatory Guide 1.142, Rev. 2. RG 1.142, Rev. 2 endorses ACI 349-97 with some additional or alternate requirements as stated in the regulatory positions. In regard to these positions stated in RG 1.142, Rev. 2, STP 3&4 is committed to following the additional requirements in RG 1.142, Rev. 2 as applied to ACI 349-97.

   Additional requirements in the DCD regarding safety-related concrete design (e.g. Table 3.8-10) are not affected by this code year change, and will be implemented in design.

   A detailed review of the differences between ACI 349-80 and ACI 349-97, as they apply to the design and analysis of safety-related ABWR standard design, has been performed. Generally, revisions provided expanded explanations of the code requirements to eliminate possible misinterpretations or to identify specific instances where the code section applies or does not apply; incorporated provisions based on more current research or experience; or expanded provisions to address new types or methods of construction that were not clearly allowed or disallowed in earlier revisions.

   The following is a summary description of the changes that may be both significant as well as applicable to the ABWR standard design, along with associated justifications for accepting the differences.

   Chapter 9 – Strength and Serviceability Requirements, contains changes in Section 9.5 pertaining to calculation of long term deflections. These changes simplify calculation of deflection magnification factors and allow for determining deflection at different time periods. As the design is not expected to be governed by deflection control, these changes will not affect ABWR standard design.

   Chapter 10 – Flexure and Axial Loads, includes changes in Section 10.6 to replace provisions that were determined to be inadequate based on more recent experience, and are, therefore, improvements. Changes in the other sections address more recent construction practices and experience, and will result in no change or more conservative design margins.

Chapter 11 – Shear and Torsion, includes a large number of changes, most of which are additional provisions or are changes based on more recent research results and experience. None of these changes will reduce design margins for ABWR standard design.

Chapter 12 – Development and Splices of Reinforcement, includes a large number of changes, most of which are provisions to address epoxy coated rebar (the ABWR does not use epoxy coated rebar) and revised provisions for reinforcement development length. These changes are based on more recent extensive research results and experience and generally result in increased lengths for development.

Chapter 21 – Special Provisions for Seismic Design, has been added in ACI 349-97 and provides requirements for analysis and design for seismic loading. These provisions are intended to improve the toughness of the structure and to assure that the integrity of the structure is retained even under inelastic deformations due to earthquake events. These provisions are based on more current research and experience, represent the state of the art at the time of the code revision, and therefore its use will result in more robust structures.

Appendix B – Steel Embedments, includes changes in ACI 349-97 based on later research. The changes in Appendix B are for the local design of embedment plates and do not affect the design of the major concrete elements. Additionally, the supplemental requirements defined in the Staff Positions in DCD Table 3.8-10 will have a larger impact on the embedment design.

Although Appendix C (Special Provisions for Impulsive and Impactive Effects) has not been revised, the additional requirements defined in Regulatory Guide 1.142, Rev. 2, Positions 10 and 11 will be included.

2. STPNOC will comply with the guidance provided in RG 1.206, Section C.I.1.9.2 and use Standard Review Plan (SRP) Section 3.8.4, Revision 2 for structures outside the scope of the ABWR DCD. According to SRP acceptance criteria 3.8.4.II.5, ANSI/AISC N690-1994 including Supplement 2 (2004) has been accepted by the staff for design, fabrication, and erection of safety-related steel structures. The mark-up for COLA Part 2, Tier 2, Table 1.8-21a is provided at the end of this response.

3. STD DEP 1.8-1, "Tier 2* Codes, Standards, and Regulatory Guide Edition Changes," includes several changes. As noted in the RAI Question, Table 1.8-21 references ASME Code, Section III, Division 2, Edition 2001 with 2003 Addenda. In addition, Table 1.8-20 changes the commitment for Regulatory Guide 1.136, "Design Limits, Load Combinations, Materials, Construction, and Testing of Concrete Containments", from Revision 2 to Revision 3. Revision 3 of Regulatory Guide 1.136 endorses ASME Code, Section III, Division 2, Edition 2001 with 2003 Addenda.

   Additional requirements in the DCD regarding containment design (e.g. Table 3.8-2) are not affected by this code year change, and will be implemented in design.

   A detailed review of the differences between the 1989 edition and the 2001 edition with 2003 addenda of the ASME Code, Section III, Division 2, as they apply to the design and analysis of safety-related ABWR containment structure, has been performed. Below is a summary of the significant Code changes along with the justifications for the Code differences as required to evaluate the acceptability of the 2001 Edition with 2003 Addenda. Note that no changes were identified to the load categories and load combinations in Sections CC-3220 and CC-3230.

Section CC-3421.8, "Brackets and Corbels", was substantially revised to incorporate the concept of shear-friction for the design of steel reinforcement (Similar to ACI 318). This change has no impact, as there are no brackets or corbels required for the ABWR containment design.

Section CC-3424, "Shear Friction", was added to provide details of the shear-friction design method (Similar to ACI 318), which is to be applied where it is appropriate to consider shear transfer across a given plane such as a potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times. This is a code enhancement, since such guidance was not available in the 1989 Edition.

In **Section CC-3431.3**, "Shear, Torsion, and Bearing", the following changes were made:

- The concrete stress service load allowable for radial shear at sections subjected to membrane tension is now 50% of the factored allowable in CC-3421.4.1(c), except that it need not be reduced below the value of 0.5 SQRT f'c. Previously, the allowable was the same as that used for the factored load allowable, except that the term Nu/Ag was to be multiplied by 2. This change results in a more conservative design.
- The service load allowable for peripheral shear at sections subjected to membrane tension remains unchanged at 50% of the factored allowable in CC-3421.6(b), except that a provision has been added that it need not be reduced below the value of 0.5 SQRT f'c. This is a very minor relaxation. It will have no significant impact on the design since the service load condition is not expected to control for peripheral shear.
- Service load requirements for design of reinforcement for brackets and corbels were modified based on shear-friction design. This change has no impact, as there are no brackets or corbels required for the ABWR containment design.

Section CC-3531, "Reinforcing Steel Requirements – General", the sentence " For service loads, the requirements are the same, except that the computed moments shall be multiplied by 2.0 and substituted for Mu in the equations" in Paragraph (b) of the 1989 Edition was deleted. This is a code correction as reinforcing steel splice and development requirements apply to both factored loads and service loads. This change has no design impact.

Section CC-3532, "Reinforcing Steel Splicing", Paragraph (d) "Butt Splices may be welded or mechanical and shall develop a tensile strength of at least 125% of the specified minimum yield strength of the bar" in the 1989 Edition was deleted. This is a code evolution. This change will have no design impact as developing a minimum of 125% of the bar yield strength in the butt splice is covered in Table CC-4333-1.

Section CC-3532.1.2, "Development Length", was revised to clarify that Paragraph (h) applies to all main reinforcement that is terminated in a tension zone. This is standard practice and has no impact.

Section CC-3532.2.3, "Development Length for Bars in Compression", a provision was added to allow reduction of development length if the bar is not fully stressed. This is a standard provision in concrete codes, including ACI 318 and ACI 349, and has no impact.

Section CC-3533.2, "Development of reinforcement for Service Loads", in the 1989 Edition was deleted. This is a code correction as reinforcing steel development length requirement applies to both factored loads and service loads. This change has no design impact.

Section CC-3535, "Concrete Crack Control", has been modified to require minimum reinforcement of 0.0020 Ag in the containment shell. Previously, 0.0012 Ag was required for shrinkage and temperature, but a larger value of 0.0021 Ag was required in areas subject to membrane tension. This change is conservative for areas away from membrane tension and similar for areas subjected to membrane tension. As the ABWR is a non-prestressed containment, the minimum reinforcing is not expected to govern in the containment shell. Regardless, this will not affect design margins.

Section CC-3570, "Containment External Anchors", is a new section that was added to address loads, displacements, analysis methods, design allowables and other design requirements associated with anchors, embedments and other attachments acting at the external surface of the Containment. This is a code improvement.

In **Section CC-3730** (Liner Anchors), **Section CC-3740** (Penetration Assemblies) and **Section CC-3750** (Brackets and Attachments), new paragraphs were added to require that anchorage forces acting on the containment shell shall be established in the Design Specification. This has no design impact.

In **Sections CC-3740** (Penetration Assemblies) and **CC-3750** (Brackets and Attachments), the requirement to reduce the allowable stress in the thru-thickness direction has been replaced with additional steel plate examination requirements to verify that steel plates meet lamination requirements in CC-4500. The additional steel plate examination provides additional assurance that the material is free of laminations.

Sections CC-3841(i) and **CC-3842.9** were added to include Category J welded joints to the list of permissible types of liner welded joints. Category J joints are those liner joints that connect the liner plate to a steel embedment that is continuous through the liner. This is a code improvement.

The following is in response to the request for an explanation of how use of the newer Edition of the ASME Code proposed by the applicant meets the provisions of NCA-1140, "Use of Code Editions, Addenda, and Cases."

The proposed change meets the provisions of Paragraphs NCA-1140 (a)(1) and (a)(2), which read as follows:

"NCA-1140 USE OF CODE EDITIONS, ADDENDA, AND CASES

(a) (1) Under the rules of this Section, the Owner or his designee shall establish the Code Edition and Addenda to be included in the Design Specifications. All items of a nuclear power plant may be constructed to a single Code Edition and Addenda, or each item may be constructed to individually specified Code Editions and Addenda.

(2) In no case shall the Code Edition and Addenda dates established in the Design Specifications be earlier than:

(a) 3 years prior to the date that the nuclear power plant construction permit application is docketed; or

(b) the latest edition and addenda endorsed by the regulatory authority having jurisdiction at the plant site at the time the construction permit application is docketed."

This change meets Article NCA-1140(a)(2)(b) because the applicant is proposing to use the 2001 Edition with 2003 Addenda of the ASME Section III, Division 2, Code, which is the latest edition and addenda endorsed by the NRC in Regulatory Guide 1.136, "Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments", Revision 3.

As a result of this RAI response, COLA Table 1.8-21a will be revised as follows:

Table 1.8-21a Codes and Standards for Site-Specific Systems

Code or Standard Number	Year	Title
American Concrete Institute (ACI)		
349	1997	Code Requirements for Nuclear Safety-Related Concrete Structures
350	2001	Code Requirements for Environmental Engineering Concrete Structures, and Commentary (ACI 350R-01)
350.1	2001	Tightness Testing of Environmental Engineering Concrete Structures, and Commentary (ACI 350.1R-01)
American Institute of Steel Construction (AISC)		
N690†	1994*	Specifications for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities * (including Supplement 2)
American Nuclear Society (ANS)		
2.8	1992	Determining Design Basis Flooding at Power Reactor Sites
3.11	2005	Determining Meteorological Information at Nuclear Facilities

RAI 03.08.05-4

QUESTION:

Follow-up to Question 03.08.05-2

In its response to Question 03.08.05-2 (letter U7-C-STP-NRC-100108, dated May 13, 2010) regarding how differential settlements were considered for site-specific seismic category I structures, the applicant provided some information. However, in order for the staff to clearly understand the amount of differential settlement values accounted for in the design of site-specific seismic category I structures, and how these values reconcile with the estimated differential settlements at the site, the applicant is requested to provide the following additional information:

1. In Part (a) of its revised response to Question 03.08.05-2, the applicant referred to COLA Part 2, Tier 2, Section 2.5S.4.10 for conservatively calculated angular distortion/tilts. The applicant provided an explanation in its response to Question 03.08.05-3 about why the calculated angular distortions/tilts may be considered acceptable. In its justification of an acceptable tilt value of 1/500 for the seismic category I structures at STP, the applicant referenced several published materials that appear to be based on observations of cracking and structural damage of commercial structures. The applicant did not provide any justification for using this information for seismic category I structures. The information included in the response does not provide any estimate of the amount of additional stresses that may be imposed on these structures as a result of the tilt. Therefore, in order for the staff to conclude that the acceptable tilt of 1/500 for the seismic category I structures at STP will not adversely impact the calculated stresses in these structures in critical areas, the applicant is requested to provide a quantitative evaluation that explicitly considers the tilt for these structures.

2. In Part (b) of its revised response to Question 03.08.05-2 on Differential Settlement due to Flexibility of Structure/Basemat and Supporting Soil, the applicant stated that the effect of settlement due to the flexibility of the structure/basemat and supporting soil is accounted for through the use of finite element analysis (FEA) in conjunction with foundation soil springs. However, the foundation subgrade modulus may vary over a wide range across the foundation footprint. It is not clear from the response if the applicant considered in the analysis the horizontal variation of foundation subgrade modulus over the entire area of the foundation. Also, it is not clear from the response how the differential settlements accounted for in the design through the FEA modeling reconcile with the calculated differential settlements in Section 2.5S.10.4 of the FSAR and the values of maximum differential settlements that the structures are designed for. Therefore, in order for the staff to complete its review of how differential settlements are accounted for in the design of site-specific seismic category I structures, the applicant is requested to provide the following additional information:

· Describe how the variation of the subgrade modulus over the foundation footprint has been considered in the analysis, and

· List in the FSAR the values of maximum differential foundation settlements for which each seismic category I structure is designed.

RESPONSE:

The following provides the response to part 1 of this RAI. The response to part 2 of this RAI will be provided in a supplemental response by October 25, 2010.

1. As noted in the response to RAI 03.08.05-3, Revision 2 (letter U7-C-STP-100108, dated May 13, 2010), the induced stresses due to flexibility of the structure/basemat and the supporting soil are accounted for through use of Finite Element Analysis (FEA) in conjunction with use of appropriate springs representing the foundation soil.

   To evaluate the induced stresses in site-specific Seismic Category I structures due to rigid body tilt, the maximum allowed rigid body tilt of 1/500 is considered. As can be seen from Figure 03.08.05-4.1, under a maximum rigid body tilt of 1/500, the structure will be subjected to additional lateral loads equal to 0.002 times the gravity loads (or 0.2% of gravity loads). All site-specific Seismic Category I structures are qualified for site-specific Safe-Shutdown Earthquake (i.e. 0.13g modified Regulatory Guide 1.60 spectra). Conservatively assuming no in-structure amplification, the minimum lateral seismic load for the design of site-specific Seismic Category I structures equals to 0.13 times the gravity loads (or 13% of gravity loads). Therefore, for STP site-specific Seismic Category I structures, the induced stresses due to 1/500 rigid body tilt about the E-W or N-S axis will be less than 1.5% (i.e. 0.2/13 = 0.015) of the stresses due to design lateral seismic loads due to N-S or E-W excitations, respectively. Note that the induced stresses due to maximum rigid body tilt of 1/500 when compared to the total governing design stresses (i.e. stresses due to all loads within the governing load combinations such as dead load and live load in combination with seismic loads) will be far less than 1.5% of the governing design stresses. Thus, the induced stresses due to maximum rigid body tilt of 1/500 are negligibly small and a maximum rigid body tilt of 1/500 is considered acceptable without any explicit evaluation.

No additional COLA revision is required as a result of this response.

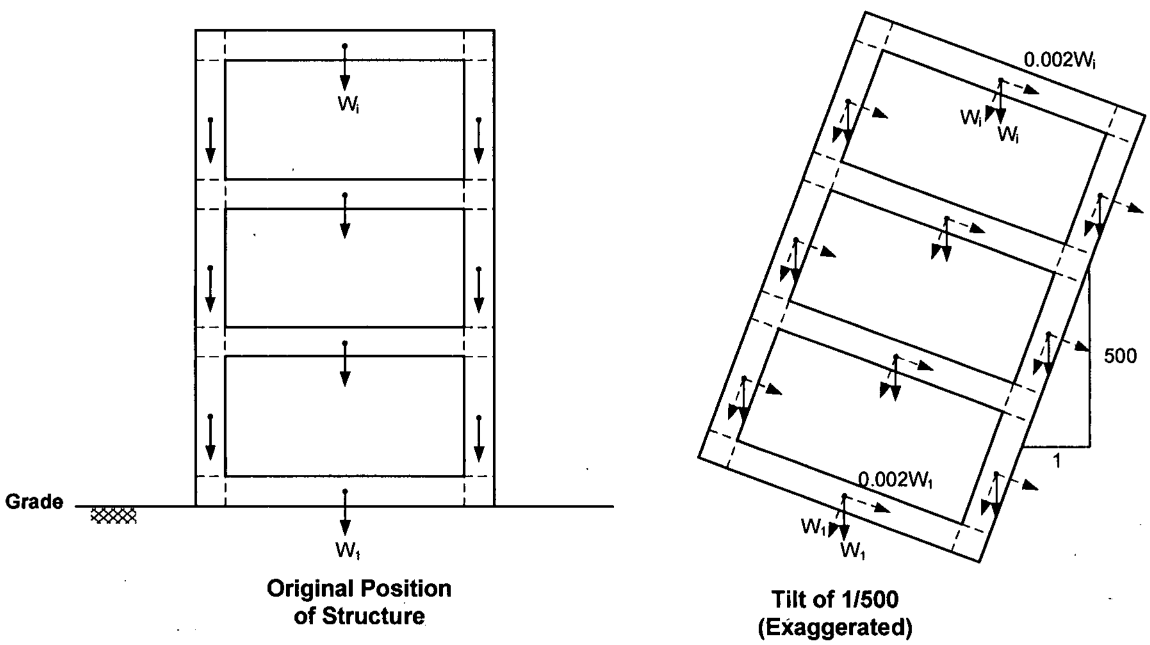


Figure 03.08.05-4.1: Induced Loads due to Rigid Body Tilt of 1/500

RAI 03.08.05-5

QUESTION:

Follow-up to Question 03.08.05-3

In its response to Question 03.08.05-3 (letter U7-C-STP-NRC-100083, dated April 14, 2010), the applicant stated that the ABWR DCD does not contain any criteria for settlement-related angular distortions/tilts. The applicant explained that its use of an acceptable tilt value of 1/500 for Category I structures is based on information from several published literature. However, the applicant did not provide any information about the amount of additional stress that may be imposed on the standard plant structures as a result of the acceptable tilt of 1/500. The applicant further stated that structural analysis and design of the structures account for the induced stresses due to structural and foundation flexibility. However, it is not clear from the response if the expected differential settlements for the standard plant structures at the STP site would be within the values of differential settlements that were accounted for in the analysis of ABWR standard plant structures. Therefore, to address COL information item 3.24, which requires that the physical properties of the site-specific subgrade medium be determined, and the settlement of foundations and structures, including seismic category I, be evaluated, the applicant is requested to:

1. Provide a quantitative evaluation of the proposed acceptance criteria for foundation tilt to demonstrate that the ABWR standard plant structures would not be adversely stressed as a result of the tilt.

2. Provide a quantitative evaluation to demonstrate that the maximum differential settlements for the ABWR standard plant structures at the STP site would be within the values accounted for in the design of these structures.

Please also update the FSAR to clearly state how this COL information item is addressed. The staff needs this information to conclude that the ABWR standard plant structures are adequate to accommodate site-specific differential settlements.

RESPONSE:

The following provides the response to part 1 of this RAI. The response to part 2 of this RAI will be provided in a supplemental response by October 25, 2010.

1. As noted in the response to RAI 03.08.05-3, Revision 2 (letter U7-C-STP-100108, dated May 13, 2010), the induced stresses due to flexibility of the structure/basemat and the supporting soil are accounted for through use of Finite Element Analysis (FEA) in conjunction with use of appropriate springs representing the foundation soil.

To evaluate the induced stresses in ABWR Standard Plant Seismic Category I structures due to rigid body tilt, the maximum allowed rigid body tilt of 1/500 is considered. As can be seen from Figure 03.08.05-5.1, under a maximum rigid body tilt of 1/500, the structure will be subjected to additional lateral loads equal to 0.002 times the gravity loads (or 0.2% of gravity loads). All ABWR Standard Plant Seismic Category I structures are qualified for 0.3g Regulatory Guide 1.60 Safe-Shutdown Earthquake spectra. Conservatively assuming no in-structure amplification, the minimum lateral seismic load for the design of ABWR Standard Plant Seismic Category I structures equals to 0.3 times the gravity loads (or 30% of gravity loads). Therefore, for ABWR Standard Plant Seismic Category I structures, the induced stresses due to 1/500 rigid body tilt about the E-W or N-S axis will be less than 0.67% (i.e. 0.2/30 = 0.0067) of the stresses due to design lateral seismic loads due to N-S or E-W excitations, respectively. Note that the induced stresses due to maximum rigid body tilt of 1/500 when compared to the total governing design stresses (i.e. stresses due to all loads within the governing load combinations such as dead load and live load in combination with seismic loads) will be far less than 0.67% of the governing design stresses. Thus, the induced stresses due to maximum rigid body tilt of 1/500 are negligibly small and a maximum rigid body tilt of 1/500 is considered acceptable without any explicit evaluation.

No additional COLA revision is required as a result of this response.

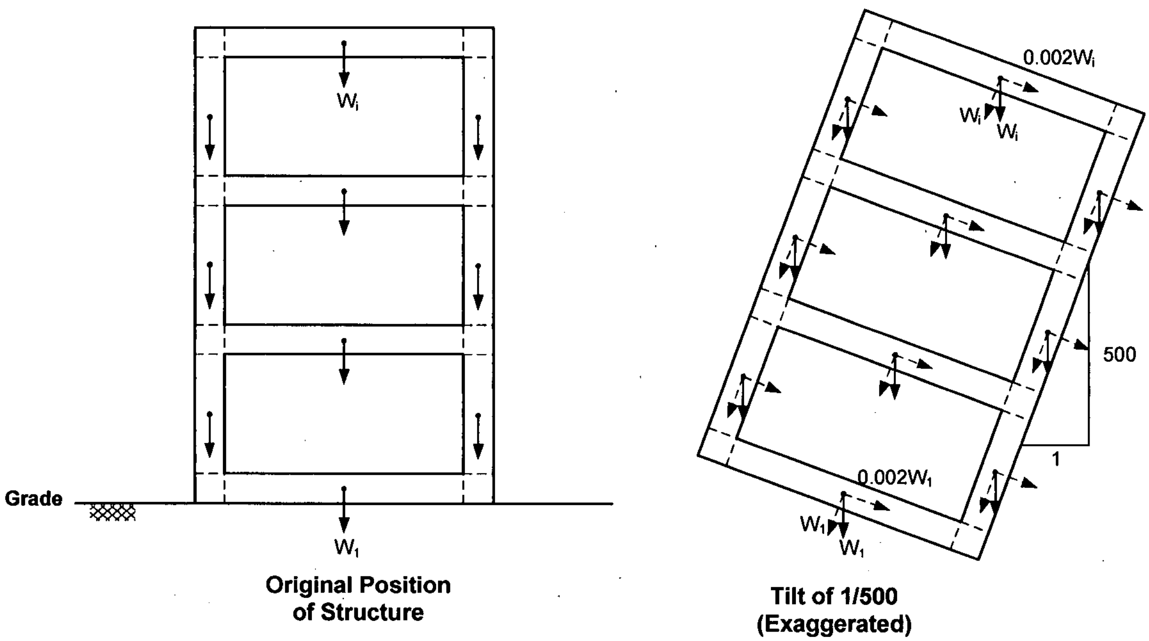


Figure 03.08.05-5.1: Induced Loads due to Rigid Body Tilt of 1/500