The Seismic Category I Forebay and UHS Makeup Water Intake Structure are reinforced concrete structures situated along the western shoreline of the Chesapeake Bay. As illustrated in Figure 3.8-4, the Forebay is connected to the CWS Makeup Water Intake Structure (Seismic Category II) and the Intake Pipes (Seismic Category I) from the north (plant reference) and the UHS Makeup Water Intake Structure from the south. The two intake pipes transport water (under gravitational head) from the Chesapeake Bay to the Forebay, which supplies water to both the CWS Makeup Water Intake Structure and the UHS Makeup Water Intake Structure. The UHS Makeup Water Intake Structure houses components associated with the UHS Makeup Water System, which provides makeup water to the Essential Service Water Cooling Tower basins for extended cooling that starts 72 hours after a design basis accident. Figure 3.8-1 shows the position of the Forebay and UHS Makeup Water Intake Structure relative to the NI.

A general area drawing of the UHS Makeup Water Intake Structure, Circulating Water Makeup Intake Structure and the Forebay is shown in Figure 9.2-4. Plan views of the UHS Makeup Water Intake Structure are shown in Figure 9.2-5 and Figure 9.2-6. A section view is shown in Figure 9.2-8.

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The Forebay is a below-grade ~~open top~~ reinforced concrete water basin, with overall dimensions of 109 ft (33.2 m) long by 89 ft (27.1 m) wide by 39 ft (11.9 m) deep, including a 5 ft (1.5 m) thick basemat. Inside dimensions of the Forebay are 100 ft (30.5 m) long by 80 ft (24.4 m) wide, with 4.5 ft (1.4 m) thick walls. The Forebay is embedded approximately 37.5 ft (11.4 m) below the nominal grade elevation of 10 ft (3.0 m), with the top of the walls at elevation 11.5 ft (3.5 m) and the top of the basemat at elevation -22.5 ft (-6.9 m).

The UHS Makeup Water Intake Structure is a reinforced concrete structure 93 ft (28.3 m) long by 58 ft (17.7 m) wide by 69 ft (21 m) high, including a 5 ft (1.5 m) thick basemat that is integrally connected with the Forebay basemat. The structure consists of a below-grade water basin 59 ft (18.0 m) long by 58 ft (17.7 m) wide by 39 ft (11.9 m) deep situated approximately 37.5 ft (11.4 m) below the nominal grade elevation of 10 ft (3.0 m) and an above-grade pump house structure situated partially above the water basin and partially over structural fill.

The five main elevations of the UHS Makeup Water Intake Structure are:

- Elevation -22.5 ft (-6.9 m): Bottom of the water basin and top of the basemat. There are four independent pump bays in the water basin, separated by reinforced concrete walls.
- Elevation 11.5 ft (3.5 m): Top of the operating deck and pump house floor, which includes four make-up water pump rooms separated by reinforced concrete walls. Each of the four make-up water pump rooms contains an air handling unit. The pump rooms are water-tight to protect against hurricane floods.
- Elevation 21.0 ft (6.4 m): Top of floor containing four makeup water traveling screens, which includes four traveling screen rooms separated by reinforced concrete walls. The rooms are elevated above probable maximum storm surge floods and the walls are water-tight to protect against hurricane floods, including surge, wave heights, and wave run-up.
- Elevation 26.5 ft (8.1 m): Top of the floor containing four UHS makeup water transformer rooms, each of which houses a transformer, and four air cooled condenser rooms, each of which houses an air cooled condenser.
- Elevation 41.5 ft (12.6 m): Top of the nominally 2 ft (0.6 m) thick, reinforced concrete roof slab.

1.142 (NRC, 2001)).

As noted in Section 3.8.4.1.9, buried pipes are located such that the top surface of the pipe is below the site-specific frost depth, with additional depth used to mitigate the effects of surcharge loads and tornado or turbine generated missiles. In lieu of depressing the pipes in the soil beyond that required for frost protection, i.e., to obviate the risk of tornado or turbine generated missile impacts, permanent protective steel plates, located at grade, may be designed.

Bending stresses in buried pipe due to surcharge loading are determined via manual calculations, treating the flexible pipe as a beam on an elastic foundation. Resulting stresses are combined with operational stresses, as appropriate.}

3.8.4.4.6 Design Report

{Design reports for the Forebay and UHS Makeup Water Intake Structure are presented in Appendix 3E.4. Design reports for Seismic Category I Buried Piping and Seismic Category I Buried Duct Banks are presented in Appendices 3E.5 and 3E.6, respectively.}

3.8.4.4.7 {Forebay and UHS Makeup Water Intake Structure

This section is added as a supplement to U.S. EPR FSAR Section 3.8.4.4.

The Forebay and UHS Makeup Water Intake Structure are reinforced concrete shear wall structures. Vertical loads are transferred to the foundation basemat through the reinforced concrete walls before being transferred to the supporting soil through bearing pressure. Lateral loads, including those that are seismically induced, are transferred to the supporting soil by the foundation basemats and below-grade walls through friction, adhesion, and passive soil pressure, if necessary.

A finite element (FE) model was created for the Seismic Category I Forebay, UHS Makeup Water Intake Structure and Seismic Category II CWS Makeup Water Intake Structure, using STAAD Pro (Version 8i). The CWS Makeup Water Intake Structure is included in the FE model since it is integrally connected to the Forebay, shown in Figure 9.2-4. Since the CWS Makeup Water Intake Structure, Forebay, and UHS Makeup Water Intake Structure share a common basemat, they are also known as the Common Basemat Intake Structures (or CBIS).

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STAAD Pro is a commercial structural engineering computer program developed by Bentley Systems, Inc. QA and QC requirements for safety-related structures are documented in the vendor's validation and verification manuals. The program is accepted for use in accordance with RIZZO's engineering department and QA procedures. The program is in compliance with the requirements of ASME NQA-1-1994 (ASME, 1994). The STAAD Pro FE model is converted to a SASSI model using ~~RIZZO computer code~~ACS SASSI, version ~~1.3a~~2.3.0, to perform soil-structure interaction (SSI) analysis. SSI analysis is discussed in Section 3.7.2. Due to the ~~SASSI~~ limitations ~~in node numbers~~of the computer code, the SASSI model has a slightly coarser mesh than the STAAD model.~~, a symmetric plan in the FE model had to be considered for the SSI analysis.~~

The STAAD Pro FE model is also used to conduct static analysis under non-seismic loads to compute the structural responses, generate results for the design of reinforced concrete structural elements, and perform static stability and bearing pressure evaluations. The finite element analysis results from the SSI analysis and the static analysis are combined to determine the reinforced concrete design forces and moments under seismic load cases.

The FE model is described in detail in Section 3.7.2.3. Figure 3.7-23 and Figure 3E-5 depicts the FE model for the static analysis of the CBIS. The entire CBIS is modeled, without assuming a symmetry plane, and the UHS MWIS is modelled in greater detail.

For the static analysis, the soil medium below the foundation basemat is represented by soil spring elements. The modulus of subgrade reaction for the soil spring elements is based on the site-specific soil properties presented in Section 2.5.4. Effects of the following loads are calculated from the static analysis: dead loads, live loads (including snow loads), hydrostatic loads, lateral earth pressure loads (including groundwater effects), buoyancy loads, wind loads, tornado loads (including wind pressure and differential pressure effects), SPH and PMH loads (including hydrostatic pressure, buoyancy, wave pressure, and concurrent wind pressure effects). Pipe reactions are considered by applying a blanket load of 50 psf to the structure.

During maintenance of the UHS Makeup Water Intake Structure, when stop logs are installed, interior or exterior below-grade cells may be empty. The exterior embedded walls, with the empty adjacent cell, are subject to lateral soil pressure, surcharge and hydrostatic pressure from a normal groundwater level of +3 ft (0.9 m) NVGD 29. This postulated maintenance condition is considered in the FE model for designing the side walls of the UHS Makeup Water Intake Structure.

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Seismic-induced hydrodynamic loads associated with the water contained in the CBIS are calculated according to the provisions of ACI 350.3-06 (ACI, 2006). Effects of the impulsive and convective components of the hydrodynamic loads are calculated in the SSI analysis by including the corresponding water mass and springs in the ACS SASSI model.

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The accelerations determined from the SSI analysis are applied to the FE model and combined with other static analyses to generate design forces and moments for load combinations involving seismic effects, in accordance with Section 3.8.4.3.2. Seismic accelerations for a particular earthquake direction are computed by adding the accelerations of three directions at a given location using the ~~Square Root of the Sum of the Squares (SRSS) method~~algebraic summation method for each point in time. Accelerations are then enveloped for a particular direction for all soil profiles (i.e., UB, BE, LB described in Section 3.7.1.3.3).

Following application of the SASSI accelerations from the three components of earthquake motions to the static model, the results are combined using the Square Root of the Sum of the Squares (SRSS) method, as described in Section 3.7.2.6. The design forces and moments from seismic and non-seismic load combinations are used to design reinforced concrete shear walls and slabs according to the provisions of ACI 349-01 (ACI, 2001a) (with supplemental guidance of Regulatory Guide 1.142 (NRC, 2001)). Results of the reinforced concrete design are provided in Appendix 3E Section 3E.4.5.

The evaluation of slabs and walls for external hazards (e.g., tornado generated missiles) is performed by local analyses, following the procedure outlined in U.S. EPR FSAR Section 3.8.4.4.1. Procedures for stability evaluation and bearing pressure calculation are discussed in Section 3.8.5.4.6.}

3.8.4.5 Structural Acceptance Criteria

The U.S. EPR FSAR includes the following COL Item in Section 3.8.4.5:

> A COL applicant that references the U.S. EPR design certification will confirm that site-specific conditions for Seismic Category I buried conduit, electrical duct banks, pipe, and pipe ducts satisfy the criteria specified in Section 3.8.4.4.5 and those

Service loads

- D + H + L + F +Fb

Severe environmental loads

- D + H + L + F +Fb + W
- D + H + L + F +Fb + SPH

Extreme environmental loads

- D + H + L + F +Fb + Wt
- D + H + L + F +Fb + E'
- D + H + L + F +Fb + PMH}

3.8.5.4 Design and Analysis Procedures

No departures or supplements.

3.8.5.4.1 General Procedures Applicable to Seismic Category I Foundations

No departures or supplements.

3.8.5.4.2 Nuclear Island Common Basemat Structure Foundation Basemat

No departures or supplements.

3.8.5.4.3 Emergency Power Generating Buildings Foundation Basemats

No departures or supplements.

3.8.5.4.4 Essential Service Water Buildings Foundation Basemats

No departures or supplements.

3.8.5.4.5 Design Report

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{Design reports for the Forebay and UHS Makeup Water Intake Structure basemats are presented in Appendix 3E.4.}

3.8.5.4.6 {Forebay and UHS Makeup Water Intake Structure Basemats

This section is added as a supplement to U.S. EPR FSAR Section 3.8.5.4.

As shown in Figure 3.7-23, the foundation basemats are part of the finite element model used for the analysis and design of the Seismic Category I Forebay and UHS Makeup Water Intake Structure. The finite element mesh of the basemats is shown in Figure 3.8-5. ~~Note that only half of the basemat is modeled because of symmetry.~~ Analysis and critical section design procedures for these structures are presented in Section 3.8.4.4.7.

To ensure the stability of the structures during various design basis events, the Common Basemat Intake Structures (CBIS) are checked for sliding, overturning, and flotation using the stability load combinations described in Section 3.8.5.3.

Static and dynamic bearing pressures are calculated and compared with the bearing capacities defined in Table 2.5-67.

<u>For the static load combinations, the STAAD model maximum bearing pressures at each node are obtained by dividing the nodal reaction (spring force) by the nodal tributary area.</u>

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~~Both average and maximum bearing pressures are determined for the STAAD Pro FE model. The average bearing pressure is determined by summing the support node reaction forces below the CBIS basemat and dividing it by the area of the basemat. Maximum bearing pressures are calculated as follows:~~

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- ~~At each support node, the nodal reaction (i.e., spring force) is divided by the number of plates connected to that node and the resulting force assigned to each of the plates connected to the node.~~
- ~~For each plate, the force contributions from its nodes are summed to yield a total reaction force for the plate.~~
- ~~The bearing pressure for a particular plate is determined by dividing the plate reaction by the area of the plate.~~
- ~~The average bearing pressure below the basemat is calculated as the average of all the bearing pressures.~~
- ~~The bearing pressure below the UHS is determined as the average of the bearing pressures for the UHS basemat area only. In a similar manner, bearing pressures are determined for the Forebay and Circulating Water Makeup Intake Structure by calculating the average bearing pressure below the particular basemat area. The maximum of these three bearing pressures are referred to as a "Maximum" in order to distinguish this pressure from the average bearing pressure for the entire basemat.~~

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Results from the SASSI analysis are used to calculate sliding forces and overturning moments for seismic loads, as described in Section 3.7.2.14.3. The loads contributing to the structural mass in the SSI analysis are used to calculate the resistance to sliding and overturning. These loads include the self weight of the structure, weight of the permanent equipment and contained water during normal operation, 25% of the design live load and 75% of the design snow load. ~~The design model in STAAD Pro is also checked for the sliding and overturning conditions for the SSE case. The STAAD Pro model for the SSE case contains the conservatively applied acceleration obtained from the SASSI SSI analysis and it does not contain the live and snow loads. The reaction forces from the STAAD Pro SSE model are used for stability evaluation and the results are reported in Table 3.8-2.~~

For the non-seismic loads, basemat reactions from STAAD Pro analysis are used to calculate sliding forces and overturning moments and results are reported in Table 3.8-2. The dead load used to calculate the resistance to sliding and overturning includes the self weight of the structures, permanent equipment and water inside structures during the normal operation, SPH and PMH conditions.

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Flotation is checked under normal operation, SPH, and PMH conditions, including the draw-down condition during a PMH event, with the water inside the CBIS at the minimum design level of -8 ft (-2.4 m). Resistance to flotation is provided by dead load.

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Sliding is checked at various sliding interfaces below the foundation basemats. The CBIS sits on top of a mud mat, which is placed directly on the in-situ soil stratum IIc (Chesapeake clay/silt). Therefore, resistance to sliding is provided by friction between the basemat and the mud mat and friction and adhesion between the mud mat and soil stratum IIc. ~~Friction (traction) between the below-grade walls and structural fill is also utilized for SSE loads.~~ Passive soil pressure is not utilized for the stability of the CBIS. The static coefficients of friction for various sliding interfaces are presented in Table 3.8-1.

Frictional resistance is reduced by the effects of any upward forces, such as upward seismic forces and buoyancy. Overturning resistance is reduced by buoyancy.

The factors of safety from aforementioned stability evaluations are compared with the minimum required factors of safety specified in U.S. EPR FSAR Table 3.8-11. The minimum required factors of safety for sliding and overturning associated with SPH and PMH are the same as those for wind and tornado, respectively. The minimum required factor of safety for flotation, including SPH and PMH conditions, is 1.1.

Results of the stability and bearing pressure evaluations are presented in Section 3.8.5.5.4.}

3.8.5.5 Structural Acceptance Criteria

The U.S. EPR FSAR includes the following COL Item in Section 3.8.5.5:

> A COL applicant that references the U.S. EPR design certification will evaluate site-specific methods for shear transfer between the foundation basemats and underlying soil for site-specific soil characteristics that are not within the envelope of the soil parameters specified in Section 2.5.4.2.

This COL Item is addressed as follows:

{For the Nuclear Island (NI) common basemat structures, Emergency Power Generating Buildings (EPGBs), and Essential Service Water Building (ESWBs), U.S. EPR FSAR Section 2.5.4.2 specifies a minimum coefficient of friction of 0.5 for interfaces between the foundation basemat and soil, or for cohesive soil cases the soil will have an undrained strength equivalent to or exceeding a drained strength of 26.6 degrees yielding a friction coefficient greater than or equal to 0.5. As identified in Table 3.8-1, the coefficient of friction for underlying interfaces is typically greater than 0.5. In those instances where the coefficient of friction is less than 0.5, there is an adhesion component providing additional resistance to movement (see Table 3.8-1). As identified in Table 2.5-54, the drained strength or drained friction angle (f') is greater than 26.6 degrees.

A site-specific sliding evaluation for SSE loads is performed to confirm the sliding stability of NI common basemat structures, EPGBs, ESWBs, NAB, AB, and Turbine Island (TI). These structures are located in the powerblock area, which will be excavated and backfilled. Mud mats are used under the basemat of each structure to facilitate construction. As described in Section 3.8.4.6.1, a waterproofing system is used to protect the NI common basemat structures, ESWBs, NAB, and AB from the low-pH groundwater, as illustrated in Figure 3.8-6. The potential sliding interfaces down to the natural soils under the NI common basemat structures, ESWBs, NAB, and AB are:

- Basemat - mud mat
- Mud mat - sand
- Sand - waterproofing membrane
- Sand - structural fill
- Structural fill - soil stratum IIb

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As described in Section 3.8.4.6.1, a dampproofing system is used for the EPGBs (and will also be used for the TI), as illustrated in Figure 3.8-7. EPGBs and TI are not exposed to low-pH groundwater and, therefore, do not require protective waterproofing and dampproofing systems. However, as a good construction practice and for defense in depth, waterproofing and dampproofing systems are applied to these structures in accordance with Sections 1805.2 and 1805.3 of the IBC 2009 (IBC, 2009). The potential sliding interfaces under the EPGBs and TI are:

- Basemat-mud mat
- Mud mat-dampproofing membrane
- Dampproofing membrane - sand
- Sand - structural fill
- Structural fill - soil stratum IIb

Frictional parameters at the various sliding interfaces are presented in Table 3.8-1. Based on these frictional parameters, factors of safety against sliding and overturning associated with the site-specific SSE loads are presented in Table 3.8-4 for the NI common basemat structures, EPGBs, and ESWBs. The minimum required factor of safety of 1.1 is achieved for all the buildings. Note that passive soil pressure is not utilized for the sliding evaluation.}

3.8.5.5.1 Nuclear Island Common Basemat Structure Foundation Basemat

The U.S. EPR FSAR included the following COL Item in Section 3.8.5.5.1:

A COL applicant that references the U.S. EPR design certification will compare the NI common basemat site-specific predicted angular distortion to the angular distortion in the relative differential settlement contours in U.S. EPR FSAR Tier 2, Figure 3.8-124 through U.S. EPR FSAR Figure 3.8-134, using methods described in U.S. Army Engineering Manual 1110-1-1904. The comparison is made through the basemat in both the east-west and north-south directions. If the predicted angular distortion of the basemat of the NI common basemat structure is less than the angular distortion shown for each of the construction steps, the site is considered acceptable. Otherwise, further analysis will be required to demonstrate that the structural design is adequate.

The COL Item is addressed as follows:

Included for Information only

{The Calvert Cliffs Unit 3 site-specific soil spring values are the same as the values used in the U.S. EPR Standard Plant settlement analysis. Due to these input values being the same as well as the construction sequence, models, methodologies, and procedures, the predicted angular distortion of the NI common basemat structure is the same for both CCNPP Unit 3 and the U.S. EPR Standard Plant.}

3.8.5.5.2 Emergency Power Generating Buildings Foundation Basemats

The U.S. EPR FSAR includes the following COL Item in Section 3.8.5.5.2:

> A COL applicant that references the U.S. EPR design certification will compare the EPGB site-specific predicted angular distortion to the angular distortion in the total differential settlement contours in Figure 3.8-135, using methods described in U.S. Engineering Manual 1110-1-1904. The comparison is made throughout the basemat in both the east-west and north-south directions. If the predicted angular distortion of the basemat of EPGB structures is less than the angular distortion shown, the site is considered acceptable. Otherwise, further analysis will be required to demonstrate that the structural design is adequate.

The COL Item is addressed as follows:

> {The Calvert Cliffs Unit 3 site-specific angular distortion values were compared to the angular distortion in the total differential settlement contours in U.S. EPR™ FSAR Tier 2, Figure 3.8-135, using methods described in U.S. Army Engineering Manual 1110-1-1904. The same models, methodologies and procedures are used as with the U.S. EPR™ Standard Plant design. The basemat area is partitioned into separate slab design areas in both the east-west and north-south directions. The maximum CCNPP Unit 3 angular distortion is less than the maximum angular distortion in every slab design area for the softest soil case in U.S. EPR™ FSAR Table 3.7.1-8; thus, the U.S. EPR™ design envelops the site.}

{The following departure is taken from U.S. EPR FSAR Section 3.8.5.5.2.

Section 2.5.4.10.2 of the U.S. EPR FSAR states that:

> "The design of Seismic Category I foundations for the U.S. EPR is based on a maximum differential settlement of ½ inch per 50 ft in any direction across the basemat."

The U.S. EPR FSAR maximum allowable differential settlement of ½ inch per 50 ft may also be expressed as a fraction, i.e., 1/1200.

According to Section 2.5.4.10.2, the estimated site-specific differential settlement is 1/1166, which is about 3% higher than the allowable value described in the U.S. EPR FSAR.

A finite element analysis of the entire EPGB structure, including CCNPP Unit 3 site-specific soil springs, indicates the maximum differential settlement within the confines of the EPGB basemat is 1/2714, or substantially less than the allowable value of the U.S. EPR FSAR. The variation of the finite element analysis differential settlement (1/2714) with the estimated differential settlement value of 1/1166 is attributed to the conventional geotechnical treatment of the foundation as a flexible plate, a condition much more conservative than the actual 6 ft thick reinforced concrete basemat.

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To verify the finite element analysis results, a manual calculation is performed for a selected beam strip (1 ft (0.3 m) wide by 6 ft (1.8 m) deep) of the EPGB basemat, plan view of which is shown in U.S. EPR FSAR Figure 3E.2-3. The beam strip is located at the centerline of the basemat and is perpendicular to the center reinforced concrete bearing wall. The selected two-span beam strip is 96 ft (29.3 m) long, with the aforementioned center wall and two parallel primary reinforced concrete bearing walls serving as pinned supports. Soil bearing pressures are applied to the beam strip and beam deflection is calculated. The calculation results confirm similar findings as the finite element analysis results, i.e., the maximum differential settlement of the EPGB basemat is substantially less than 1/1200.

To further evaluate the effects of the higher site-specific differential settlement, a finite element analysis of the entire EPGB is performed to evaluate the effect of a more conservative overall building tilt of L/550, where L is the least basemat dimension. For this analysis:

- Spring stiffnesses are adjusted until a tilt of L/550 is achieved.
- The elliptical distribution of soil springs is maintained.
- Soil spring stiffnesses along the centerline of the basemat (perpendicular to the direction of tilt) are retained.
- Adjustment is made to all other springs as a function of the distance from the basemat centerline.

The finite element analysis results show that increase in EPGB basemat design moment based on the more conservative differential settlement value of 1/550 (based on the overall tilt) is less than 3% of the U.S. EPR FSAR maximum design moment. Therefore, EPGB basemat is structurally adequate to resist the increased moments.}

3.8.5.5.3 Essential Service Water Buildings Foundation Basemats

The U.S. EPR FSAR includes the following COL Item in Section 3.8.5.5.3:

> A COL applicant that references the U.S. EPR design certification will compare the ESWB site-specific predicted angular distortion to the angular distortion in the total differential settlement contours in U.S. EPR FSAR Tier 2, Figure 3.8-136, using methods described in U.S. Army Engineering Manual 1110-1- 1904. The comparison is made throughout the basemat in both the east-west and north-south directions. If the predicted angular distortion of the basemat of ESWB structures is less than the angular distortion shown, the site is considered acceptable. Otherwise, further analysis will be required to demonstrate that the structural design is adequate.

The COL Item is addressed as follows:

> { TBD }

{The following departure is taken from U.S. EPR FSAR Section 3.8.5.5.3.

U.S. EPR FSAR Section 2.5.4.10.2 states that:

> "The design of Seismic Category I foundations for the U.S. EPR is based on a maximum differential settlement of ½ inch per 50 ft in any direction across the basemat."

The U.S. EPR FSAR maximum allowable differential settlement of ½ inch per 50 ft may also be expressed as a fraction, i.e., 1/1200.

Included for Information only

According to Section 2.5.4.10.2, the maximum site-specific differential settlement is 1/845, which exceeds the allowable value specified in the U.S. EPR FSAR.

A finite element analysis of the entire ESWB structure, including CCNPP Unit 3 site-specific soil springs, indicates the maximum differential settlement within the confines of the ESWB basemat is 1/1417, or less than the allowable value of the U.S. EPR FSAR. The variation of the finite element analysis differential settlement (1/1417) with the estimated differential settlement value of 1/845 is attributed to the conventional geotechnical treatment of the foundation as a flexible plate, a condition much more conservative than the actual 6 ft thick reinforced concrete basemat.

To verify the finite element analysis results, a manual calculation is performed for a selected beam strip (1 ft (0.3 m) wide by 6 ft (1.8 m) deep) of the ESWB basemat, plan view of which is shown in U.S. EPR FSAR Figure 3E.3-3. The beam strip is located at the centerline of the basemat and is perpendicular to the reinforced concrete bearing wall separating the two cooling towers. The selected two-span beam strip extends for the length of the two cooling towers, with the aforementioned divider wall and two parallel reinforced concrete bearing walls serving as pinned supports. Soil bearing pressures are applied to the beam strip and beam deflection is calculated. The calculation results confirm similar findings as the finite element analysis results, i.e., the maximum differential settlement of the ESWB basemat is less than 1/1200.

To further evaluate the effects of the higher site-specific differential settlement, a finite element analysis of the entire ESWB is performed to evaluate the effect of a more conservative overall building tilt of L/600, where L is the least basemat dimension. For this analysis:

- Spring stiffnesses are adjusted until a tilt of L/600 is achieved.
- The elliptical distribution of soil springs is maintained.
- Soil spring stiffnesses along the centerline of the basemat (perpendicular to the direction of tilt) are retained.
- Adjustment is made to all other springs as a function of the distance from the basemat centerline.

The finite element analysis results show that increase in the ESWB basemat design moments based on the more conservative differential settlement value of 1/600 (based on the overall tilt) is less than 5% of the U.S. EPR FSAR maximum design moments. So, the ESWB basemat is structurally adequate to resist the increased moments.}

3.8.5.5.4 {Forebay and UHS Makeup Water Intake Structure Basemats

This section is added as a supplement to U.S. EPR FSAR Section 3.8.5.5.

Acceptance criteria for reinforced concrete design of basemat critical sections are described in Section 3.8.4.5.

Stability and bearing pressure of the CBIS are evaluated following the procedures presented in Section 3.8.5.4.6. As reported in Table 3.8-2, factors of safety from various stability load combinations show that the minimum required values are achieved. Therefore, the CBIS are stable under various design conditions.

The average bearing pressures across the CBIS basemat and maximum localized pressures for each load combination are provided in ~~Maximum soil bearing pressures under the CBIS foundations are provided in~~ Table 3.8-3.

Static Load Combinations

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The bearing pressures for the static load combinations are obtained from the STAAD model.

The bearing capacity as reported in Table 2.5-67 is associated with the global soil failure underneath the foundation (general shear failure) rather than a local failure such as the failure of a soil element at a corner of the foundation. Therefore, the local maximum bearing pressure is not comparable to the bearing capacity reported in Table 2.5-67.

In order to make a relevant comparison, the following three steps are implemented:

1) Calculation of the resultant foundation load and its corresponding eccentricity that is equivalent to the bearing pressure distribution each load combination

2) Determination of the reduced area (effective area) due to eccentricity.

3) Computation of the increased average bearing pressure as the ratio of the total vertical load to the reduced area.

The reduced area or effective area calculated based on the eccentricity is at least 65% of the overall area. To be conservative, a reduction of 50% in the area of the CBIS is considered in the calculation of the average bearing pressure. The increased average bearing pressures corresponding to the 50% reduction in the area are shown in Table 3.8-3and these are lower than the bearing capacity.

Seismic Load Combinations

For the seismic load combination (D+L+F+E'), the static bearing pressures are summed with the seismic bearing pressures. The STAAD model is not used to evaluate seismic bearing pressures, since it is too conservative to assume maximum accelerations for all nodes to occur simultaneously. Instead, results from the SSI SASSI analysis are used to evaluate the seismic bearing pressures.

For the evaluation of seismic bearing pressures, average bearing pressures are obtained for the part of the foundation that is not subjected to uplift as follows:

1) For a given time step, the nodal net vertical pressure (seismic vertical pressure from SASSI+static vertical pressure from PLAXIS 3D) is obtained.

2) If the nodal net pressure is compressive, the pressure is multiplied with the nodal tributary area to get the nodal compressive force; negative nodal pressures are not accounted for.
3) The total compressive forces from all nodes that are in compression are summed, and divided by the area that is under compression.

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The seismic bearing capacity check is conducted for the following time steps:

1) The time step of maximum uplift, which represents the smallest area subjected to compression
2) The time step at which the compressive pressure as defined above is maximum.
3) The time step at which the overturning factor of safety is minimum
4) The time step at which the sliding factor of safety is minimum.

These time steps are the critical time steps in terms of bearing capacity check.

In addition to checking for average seismic bearing pressures, all local seismic bearing pressures are also checked at all time steps at all locations.

The SASSI simulations for all three soil cases are conducted for the operational water level and for both SSE and OBE conditions. In addition, seismic stability is checked for the maintenance and the maximum water level cases with the BE soil profile and SSE conditions.

The maximum average seismic bearing pressure is less than 4.0 ksf based on the area that is in compression. Similar to the static case, a 50% reduction to the area in compression (not the entire CBIS area) is applied to account for eccentricity, resulting in an average pressure of 8.0 ksf, which is lower than the seismic bearing capacity.

The maximum local bearing pressure, when all time steps and all cases are considered, is 18.6 ksf. For the 558 CBIS basemat solid elements checked and for more than 8000 time steps, the local bearing pressures are below 17.6 ksf except on one corner element at two time steps.

Average seismic bearing pressures the CBIS basemat (Table 3.8.3) are below the seismic bearing capacity.

The calculated maximum bearing pressures are smaller than the bearing capacities presented in Table 2.5-67 under both static and dynamic conditions.

Differential settlement across the CBIS is within the U.S. EPR FSAR differential settlement criterion of 1/1200.}

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

No departures or supplements.

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Table 3.8-2— {Stability Evaluation Results for the CBIS}

	Factors of Safety (FOS)		
Load Combination (LC)	**Sliding**	**Overturning**	**Flotation**
D + H + W ~~D + H + W~~	106~~188~~	2.1~~1.84~~	-~~-~~
D + H + Wt~~D + H + Wt~~	11.9~~23.4~~	1.6~~1.83~~	-~~-~~
D+H+E~~D + H + E'(1)(2)~~	1.1~~1.41~~	1.9~~2.83~~	-~~-~~
D+F'(1)~~D+H+E(1)(3)~~	-~~1.29~~	-~~1.26~~	1.33~~-~~
D + H + PMH~~D+F'(4)~~	28.1~~-~~	1.2~~-~~	-~~1.83~~
D + H + SPH~~D + H + PMH~~	66.4~~7.97~~	1.5~~1.69~~	-

Notes:

~~(1)~~ Factor of safety against flotation (D+F') is governed by the PMH draw-down condition.~~Friction (traction) between side wall and backfill is utilized.~~

~~(2) Factors of Safety computed from SASSI SSI analysis.~~

~~(3) Factors of Safety computed from STAAD analysis. Due to the conservatism in the SSE accelerations applied, the SASSI analysis results will be more accurate and should be used. The STAAD values are given for comparison purposes only.~~

~~(4)~~(1) ~~The factor of safety against flotation (D+F') is governed by the PMH draw-down condition.~~

Table 3.8-3— {Bearing Capacity Evaluation Results for the CBIS}

Load Combination	Bearing pressure (ksf) Average	Maximum
D + L + F	2.10	5.04
D + L + F + W	2.10	5.03
D + L + F + Wt	2.15	3.77
D + L + F + E'	1.72	5.67
D + L + F + PMH	2.78	4.94

Notes:

1. Maximum bearing pressures occur below the UHS MWIS. The maximum bearing pressure is determined as the average pressure below the UHS MWIS.
   Static and dynamic bearing capacities are 12 ksf and 18 ksf, respectively (See Table 2.5-67).

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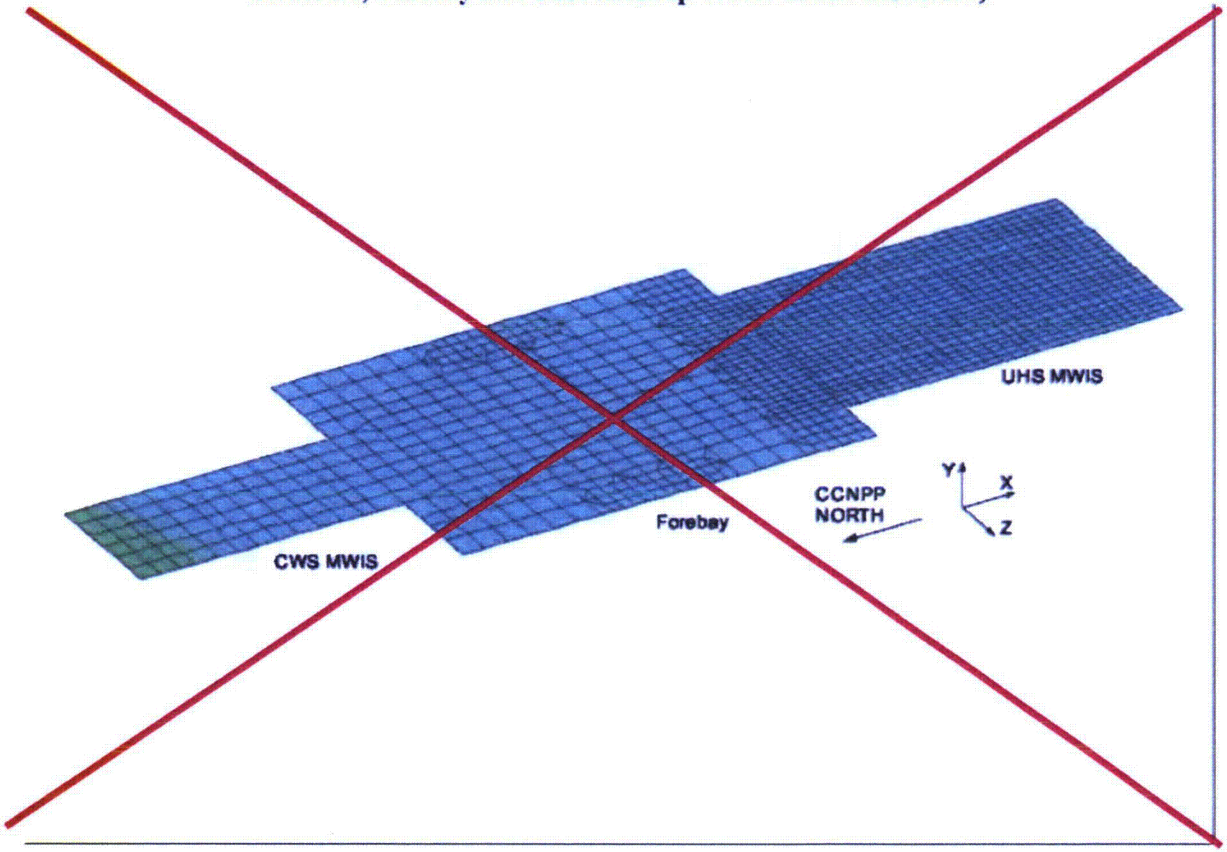
Table 3.8-3— {Bearing Capacity Evaluation Results for the CBIS}

	LOAD COMBINATION	MAXIMUM LOCAL PRESSURE (KSF)	AVERAGE BEARING PRESSURE (1) (KSF)	BEARING CAPACITY (KSF)
Static	D +L +F	14.8	5.1	11.7
	D +L +F+W	14.9	5.0	
	D +L +F+Wt	15.6	4.9	
	D +L +F+SPH	14.9	4.9	
	D +L +F+PMH	16.9	4.7	
Seismic	D+L+F+E'	18.6	8.0(2)	17.6

(1) Effective area of the foundation resisting the load is assumed as the 50% of the CBIS basemat area.
(2) Effective area of the foundation resisting the load is assumed as the 50% of the CBIS area that is in compression.

Figure 3.8-5— {Isometric View of the Basemat Finite Element Mesh (STAAD Pro Static Analysis Model) for the CWS Makeup Water Intake Structure, Forebay and UHS Makeup Water Intake Structure}



Insert for Figure 3.8-5

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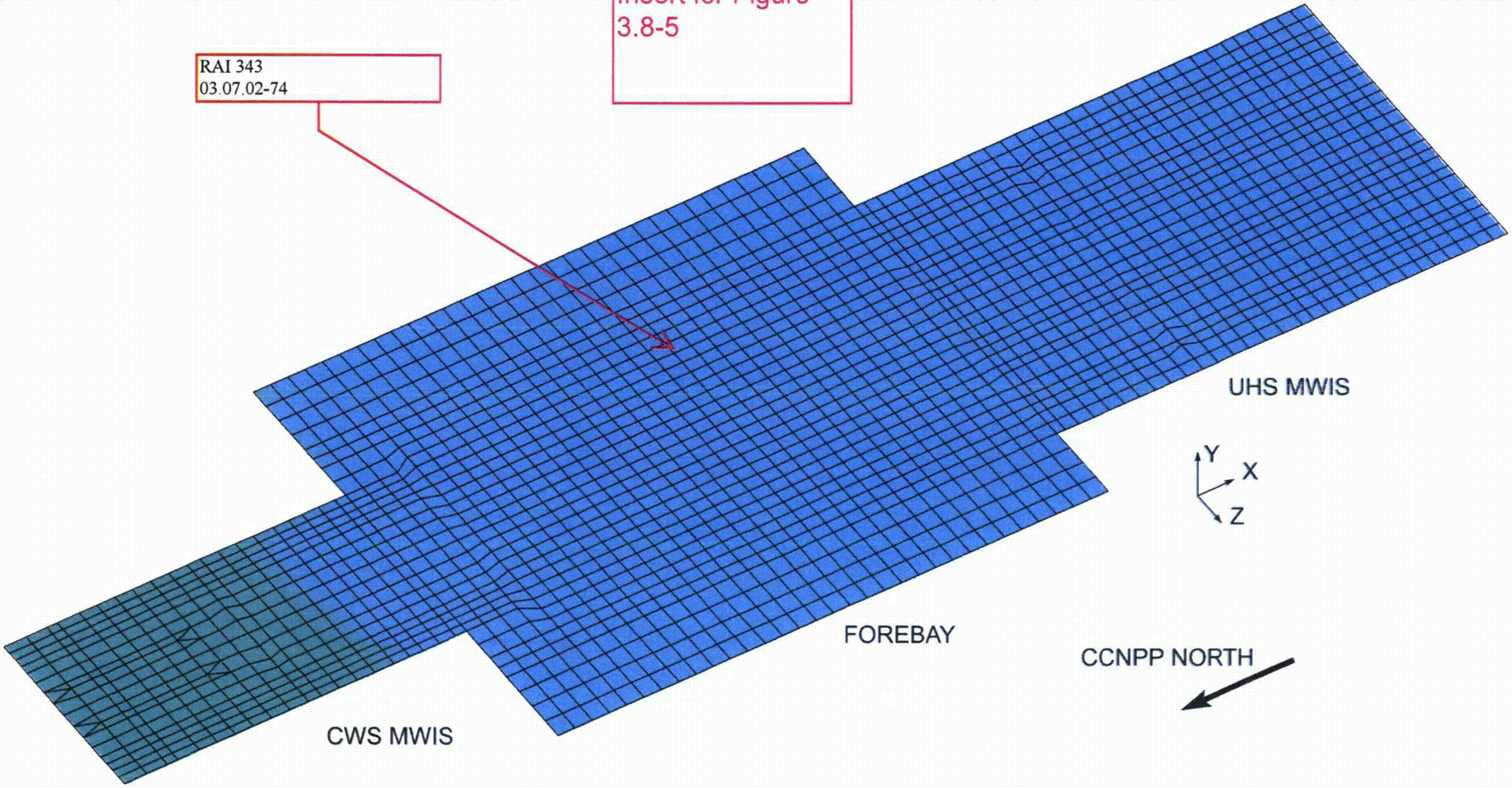

Table 3E-1— {Demand and Capacity for In-Plane Shear}

Section	Load [a] Combination	Vu [b] (kip)	φVc [c] (kip)	D/C [d]
Forebay Long Wall	Normal	2987	11725	0.25
	Wind	3038	11731	0.26
	SSE	1941	11062	0.18
	Tornado	1567	11231	0.14
	PMH	2087	11161	0.19
	SPH	2267	11737	0.19
UHS MWIS Water Basin Side Wall	Normal	4129	8170	0.51
	Wind	4138	8161	0.51
	SSE	2912	7852	0.37
	Tornado	2281	7900	0.29
	PMH	3365	8127	0.28
	SPH	2304	5532	0.42
UHS MWIS Pump House Side Wall	Normal	241	819	0.29
	Wind	253	820	0.31
	SSE	447	822	0.54
	Tornado	165	801	0.21
	PMH	279	790	0.35
	SPH	44	503	0.09

Notes:
(a) Load combinations are defined in Section 3E.4.3
(b) Vu = Maximum in-plane shear demand
(c) φVc = Nominal in-plane shear strength due to concrete as defined in Section 3E.4.4
(d) D/C = Demand/Capacity, i.e. Vu/φVn

Insert for new Table 3E-1

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Section	Load [(a)] Combination	Vu [(b)] (kip)	φVc [(c)] (kip)	D/C [(d)]
Forebay Long Wall	Normal	1006	40700	0.02
	Wind	1189	40700	0.03
	SSE	2770	40700	0.07
	Tornado	441	40700	0.01
	PMH	1208	40700	0.03
	SPH	1149	40700	0.03
UHS MWIS Water Basin Side Wall	Normal	2880	13399	0.22
	Wind	2895	13402	0.22
	SSE	2725	20885	0.13
	Tornado	2027	13244	0.15
	PMH	1993	13342	0.15
	SPH	2776	13403	0.21
UHS MWIS Pump House Side Wall	Normal	137	6751	0.02
	Wind	69	3895	0.02
	SSE	308	3895	0.08
	Tornado	106	6751	0.02
	PMH	270	3895	0.07
	SPH	238	6751	0.04

Notes:

(a) Load combinations are defined in Section 3E.4.3 (b) Vu = Maximum in-plane shear demand

(c) φVc = Nominal in-plane shear strength due to concrete as defined in Section 3E.4.4

(d) D/C = Demand/Capacity, i.e. Vu/φVn

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Table 3E-2— {Demand and Capacity for Out-of-Plane Shear}

Section	Load (a) Combination	Vu (b) (kip)	φVc (c) (kip)	D/C (d)
Common Basemat	Normal	6208	8154	0.76
	Wind	6218	8152	0.76
	SSE	3209	4198	0.76
	Tornado	2411	4242	0.57
	PMH	2295	4241	0.54
	SPH	6815	9027	0.76
Forebay Long Wall	Normal	6992	7288	0.96
	Wind	7005	7285	0.96
	SSE	5194	7122	0.73
	Tornado	5527	7320	0.76
	PMH	6263	7304	0.86
	SPH	5893	7745	0.76
UHS MWIS Water Basin Side Wall	Normal	1900	5336	0.36
	Wind	1909	5333	0.36
	SSE	2092	5235	0.40
	Tornado	915	5251	0.17
	PMH	1330	5322	0.25
	SPH	765	5772	0.13
UHS MWIS Pump House Side Wall	Normal	81	470	0.17
	Wind	85	469	0.18
	SSE	67	405	0.16
	Tornado	60	468	0.13
	PMH	190	468	0.41
	SPH	92	585	0.16

Notes:
(a) Load combinations are defined in Section 3E.4.3
(b) Vu = Maximum out-of-plane shear demand
(c) φVc = Nominal out-of-plane shear strength due to concrete as defined in Section 3E.4.4
(d) D/C = Demand/Capacity, i.e. Vu/φVc

Section	Load (a) Combination	Vu (b) (kip)	φVc (c) (kip)	D/C (d)
Common Basemat	Normal	5184	8955	0.58
	Wind	5174	8955	0.58
	SSE	3262	8470	0.39
	Tornado	3771	8792	0.43
	PMH	3129	8752	0.36
	SPH	5466	8966	0.61
Forebay Long Wall	Normal	5173	7676	0.67
	Wind	5160	7674	0.67
	SSE	2937	7566	0.39

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	Tornado	3594	7588	0.47
	PMH	2697	7524	0.36
	SPH	5292	7689	0.69
UHS MWIS Water Basin Side Wall	Normal	678	3515	0.19
	Wind	676	3513	0.19
	SSE	861	3437	0.25
	Tornado	434	3470	0.13
	PMH	334	2075	0.16
	SPH	378	2083	0.18
UHS MWIS Pump House Side Wall	Normal	17	685	0.03
	Wind	18	700	0.03
	SSE	46	669	0.07
	Tornado	49	1181	0.04
	PMH	576	1186	0.49
	SPH	437	1200	0.36

Notes:

(a) Load combinations are defined in Section 3E.4.3 (b) Vu = Maximum out-of-plane shear demand

(c) φVc = Nominal out-of-plane shear strength due to concrete as defined in Section 3E.4.4

(d) D/C = Demand/Capacity, i.e. Vu/φVc

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Table 3E-3— {Demand and Capacity for Combined Moment and Axial Force}

Section Direction	Load [a] Combination	Mu [b] (kip-ft)	Pu [c] (kip)	φMn [d] (kip-ft)	φPn [e] (kip)	D/C [f]
		(a) CBIS Common Basemat (5 ft thick) (for areas where 1 layer of #11 @ 6" each face is required)				
N-S	Normal	663	200	1079	1908	0.61
	Wind	671	195	1071	1908	0.63
	SSE	358	-8	730	-183	0.49
	Tornado	397	25	693	164	0.57
	PMH	450	236	1131	1908	0.40
	SPH	482	-18	691	-26	0.70
E-W	Normal	457	62	877	1908	0.52
	Wind	456	61	876	1908	0.52
	SSE	145	44	824	1908	0.18
	Tornado	267	66	884	1908	0.30
	PMH	269	73	897	1908	0.30
	SPH	96	-148	173	-266	0.56
		(b) CBIS Common Basemat (5 ft thick) (for areas where 2 layers of #11 @ 6" each face is required)				
N-S	Normal	663	200	1695	2103	0.39
	Wind	671	195	1690	2103	0.40
	SSE	358	-8	1432	-519	0.25
	Tornado	397	25	1396	501	0.28
	PMH	450	236	1731	2103	0.26
	SPH	123	255	998	2075	0.12
E-W	Normal	1069	51	1516	2101	0.71
	Wind	1070	51	1516	2101	0.71
	SSE	499	44	1507	2103	0.33
	Tornado	671	59	1526	2103	0.44
	PMH	741	68	1538	2103	0.48
	SPH	-1070	110	1673	172	0.64
		(c) Forebay Long Wall (4.5 ft thick) (for areas where 1 layer of #11 @ 6" each face is required)				
Vertical	Normal	540	49	735	1737	0.74
	Wind	541	485	734	1737	0.74
	SSE	205	15	685	1737	0.30
	Tornado	349	47	732	1737	0.48
	PMH	362	43	727	1737	0.50
	SPH	667	79	800	95	0.83
Horizontal	Normal	333	37	607	176	0.55
	Wind	336	38	606	175	0.55
	SSE	194	42	703	1737	0.28
	Tornado	180	21	639	254	0.28
	PMH	209	21	640	239	0.33

Table 3E-3— {Demand and Capacity for Combined Moment and Axial Force}

Section Direction	Load (a) Combination	Mu (b) (kip-ft)	Pu (c) (kip)	φMn (d) (kip-ft)	φPn (e) (kip)	D/C (f)
	SPH	347	-39	542	-60	0.6[illegible]
	(d) Forebay Long Wall (4.5 ft thick) **(for areas where 2 layers of #11 @ 6" each face are required)**					
Vertical	Normal	1106	57	1341	1803	0.82
	Wind	1106	56	1341	1803	0.82
	SSE	473	18	1299	1932	0.36
	Tornado	696	55	1339	1932	0.52
	PMH	754	50	1334	1932	0.56
	SPH	1104	95	1354	116	0.82
Horizontal	Normal	775	252	814	274	0.95
	Wind	782	254	811	270	0.96
	SSE	285	-91	1113	-538	0.26
	Tornado	441	211	891	454	0.50
	PMH	544	245	828	398	0.66
	SPH	4[illegible]5	-359	498	-430	0.83
	(e) UHS MWIS Water Basin Side Wall (4 ft thick) **(1 layer of #11 @ 9" each face)**					
Vertical	Normal	170	37	337	136	0.50
	Wind	170	38	336	136	0.50
	SSE	172	-81	264	-135	0.65
	Tornado	132	23	360	157	0.37
	PMH	103	32	345	172	0.30
	SPH	96	17	543	96	0.18
Horizontal	Normal	48	80	266	200	0.40
	Wind	49	81	264	200	0.41
	SSE	1[illegible]4	-34	342	-166	0.33
	Tornado	63	48	318	193	0.25
	PMH	40	54	309	205	0.26
	SPH	184	-53	270	-78	0.68
	(f) UHS MWIS Pump House Side Wall (2 ft thick) **(1 layer #9 @ 9" each face)**					
Vertical	Normal	13	-18	109	-131	0.17
	Wind	13	-20	107	-130	0.18
	SSE	15	-56	81	-129	0.18
	Tornado	36	-15	111	-107	0.32
	PMH	29	-34	98	-114	0.30
	SPH	27	-26	71	-67	0.38
Horizontal	Normal	6	-74	66	-138	0.53
	Wind	6	-75	64	-138	0.55
	SSE	10	-68	71	-134	0.51
	Tornado	24	-46	89	-119	0.38

Table 3E-3— {Demand and Capacity for Combined Moment and Axial Force}

Section Direction	Load (a) Combination	Mu (b) (kip-ft)	Pu (c) (kip)	φMn (d) (kip-ft)	φPn (e) (kip)	D/C (f)
	PMH	16	-57	80	-128	0.4[illegible]
	SPH	7	-76	12	-131	[illegible].58
(g) UHS MWIS Water Basin Walls (4 ft thick) (2 layers of #11 @ 6" in pairs each face)						
Vertical	Normal	175	-41	1603	-378	0.11
	Wind	175	-42	1594	-384	0.11
	SSE	20	-144	218	123[illegible]	0.09
	Tornado	105	-26	1582	-[illegible]92	0.07
	PMH	116	-35	1498	-446	0.08
	SPH	117	-46	1561	-405	0.11
Horizontal	Normal	923	-443	1215	-583	0.76
	Wind	928	-447	1213	-585	0.77
	SSE	180	-73	1296	-528	0.14
	Tornado	570	-272	1[illegible]19	-581	0.47
	PMH	6[illegible]7	-306	1207	-589	0.52
	SPH	942	-459	1207	-589	0.78

Notes:
(a) Load combinations are defined in Section 3E.4.3
(b) Mu = Bending moment demand
(c) Pu = Axial force demand (positive for compression)
(d) φMn = Bending moment capacity
(e) φPn = Axial force capacity
(f) D/C = Demand/capacity, larger of Mu/φMn and Pu/φPn

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Table 3E-3— {Demand and Capacity for Combined Moment and Axial Force}

Section Direction	Load [a] Combination	Mu [b] (kip-ft)	Pu [c] (kip)	φMn [d] (kip-ft)	φPn [e] (kip)	D/C [f]
(a) CBIS Common Basemat (5 ft thick) (for areas where 1 layer of #11 @ 6" each face is required)						
N-S	Normal	-277	31	-798	1909	0.35
	Wind	-276	31	-798	1909	0.35
	SSE	-273	0	-747	-223	0.37
	Tornado	-247	84	-889	1909	0.28
	PMH	-21	-255	-199	-329	0.78
	SPH	-331	76	-875	1909	0.38
E-W	Normal	-474	322	-1734	2104	0.27
	Wind	19	-178	371	-330	0.54
	SSE	-5	-109	-523	-335	0.33
	Tornado	303	74	894	1909	0.34
	PMH	-5	-109	-523	-335	0.33
	SPH	19	-171	387	-329	0.52
(b) CBIS Common Basemat (5 ft thick) (for areas where 2 layers of #11 @ 6" each face is required)						
N-S	Normal	19	-176	374	-330	0.54
	Wind	-446	290	-1714	2104	0.26
	SSE	-122	-6	-1399	-625	0.09
	Tornado	-367	238	-1673	2104	0.22
	PMH	-148	-375	-666	-614	0.61
	SPH	-506	249	-1684	2104	0.30
E-W	Normal	885	104	1590	2098	0.56
	Wind	883	104	1590	2099	0.56
	SSE	689	0	1460	2104	0.47
	Tornado	641	68	1545	2104	0.42
	PMH	216	-494	417	-584	0.85
	SPH	941	107	1593	2089	0.59
(c) Forebay Walls (4.5 ft thick) (for areas where 2 layers of #11 @ 6" each face is required)						
Vertical	Normal	-907	82	-1321	1878	0.69
	Wind	901	77	1316	1880	0.68
	SSE	662	49	1289	1932	0.51
	Tornado	-664	59	-1298	1932	0.51
	PMH	506	33	1273	1932	0.40

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Table 3E-3— {Demand and Capacity for Combined Moment and Axial Force}

	SPH	959	78	1317	1858	0.73
Horizontal	Normal	184	-262	814	-589	0.45
	Wind	184	-262	814	-589	0.45
	SSE	-533	55	-1347	1932	0.40
	Tornado	122	-193	940	-619	0.31
	PMH	100	-174	976	-629	0.28
	SPH	206	-275	791	-578	0.48

(d) Forebay Walls (4.5 ft thick)

(for areas where 3H+2V layers of #11 @ 6" each face are required)[9]

Vertical	Normal	-441	69	-1308	1932	0.34
	Wind	-443	70	-1309	1932	0.34
	SSE	242	42	1282	1932	0.19
	Tornado	324	43	1283	1932	0.25
	PMH	254	30	1271	1932	0.20
	SPH	-462	73	-1312	1932	0.35
Horizontal	Normal	169	-152	1588	-933	0.16
	Wind	172	-154	1584	-932	0.17
	SSE	462	49	1868	2127	0.25
	Tornado	122	-107	1663	-956	0.11
	PMH	192	-531	939	-922	0.58
	SPH	181	-172	1555	-928	0.19

(e) UHS MWIS Water Basin Walls and and El+11.5' Floor (4 ft thick)

(1 layer of #11 @ 9" each face)

Vertical or E-W	Normal	72	1407	731	1501	0.94
	Wind	80	1332	814	1501	0.89
	SSE	106	1034	1034	1501	0.69
	Tornado	50	974	1062	1501	0.65
	PMH	306	814	1113	1501	0.54
	SPH	39	1208	923	1501	0.80
Horizontal or N-S	Normal	40	-164	119	-204	0.80
	Wind	34	-153	139	-208	0.74
	SSE	6	-128	185	-222	0.58
	Tornado	28	-113	212	-211	0.54
	PMH	-276	-60	-299	-74	0.92
	SPH	41	-133	177	-204	0.65

(f) UHS MWIS Water Basin Walls (4 ft thick)

(2 layers of #11 @ 6" in pairs each face)

Vertical	Normal	-28	-483	-1361	-1335	0.36
	Wind	-25	-468	-1383	-1337	0.35

Table 3E-3— {Demand and Capacity for Combined Moment and Axial Force}

	SSE	-29	-411	-1466	-1334	0.31
	Tornado	-19	-332	-1581	-1339	0.25
	PMH	-36	-513	-1317	-1331	0.39
	SPH	-21	-439	-1426	-1338	0.33
Horizontal	Normal	809	-159	1944	-884	0.42
	Wind	784	-150	1957	-898	0.40
	SSE	-4	-150	-1958	-1347	0.11
	Tornado	650	-136	1979	-979	0.33
	PMH	478	-80	2065	-1079	0.23
	SPH	885	-175	1919	-838	0.46

(g) UHS MWIS Walls, Floors and Roof (2 ft thick)

(1 layer #9 @ 9" each face)

Vertical or E-W	Normal	-28	10	-128	769	0.22
	Wind	-29	9	-127	769	0.23
	SSE	11	-27	102	-132	0.20
	Tornado	-19	1	-123	769	0.16
	PMH	42	16	131	769	0.32
	SPH	-34	41	-146	769	0.23
Horizontal or N-S	Normal	-116	226	-231	768	0.50
	Wind	-113	224	-230	768	0.49
	SSE	-3	-40	-93	-141	0.28
	Tornado	-89	166	-201	769	0.45
	PMH	0	-51	-85	-144	0.36
	SPH	-121	246	-239	767	0.51

Notes:

(a) Load combinations are defined in Section 3E.4.3

(b) Mu = Bending moment demand

(c) Pu = Axial force demand (positive for compression)

(d) φMn = Bending moment capacity

(e) φPn = Axial force capacity

(f) D/C = Demand/capacity Ratio, Mu/φMn and Pu/φPn

Table 3E-4— {Demand and Capacity for Shear Friction}

Section	Load (a) Combination	Nu (b) (kip)	Vu (c) (kip)	φVn (d) (kip)	D/C (e)
Forebay Long Wall (2 layers of #11@6")	Normal	3929	2987	44204	0.07
	Wind	3878	3650	44204	0.08
	SSE	856.4	1974	44204	0.04
	Tornado	4521	[illegible]	44204	0.04
	PMH	4218	2087	44204	0.05
	SPH	-7868	2267	44204	0.05
UHS MWIS Water Basin Side Wall (2 layers of #11@9")	Normal	[illegible]	4129	32710	0.13
	Wind	3107	4138	32710	0.13
	SSE	1239	2979	32710	0.09
	Tornado	1572	2281	32710	0.07
	PMH	2910	2265	32710	0.07
	SPH	-5257	4285	31078	0.14
UHS MWIS Pump House Side Wall (1 layer of #9@9")	Normal	202	241	2250	0.11
	Wind	184	253	2250	0.11
	SSE	[illegible]	447	2250	0.20
	Tornado	154	165	2250	0.07
	PMH	-151	279	2250	0.12
	SPH	-332	[illegible]	2250	0.15

Notes:
(a) Load combinations are defined in Section 3E.4.3
(b) Nu = Normal force on friction interface (positive for tension)
(c) Vu = Shear demand, vector sum of in-plane and out-of-plane shear
(d) φVn = Nominal shear friction strength
(e) D/C = Demand/Capacity, i.e. Vu/φVn

Section	Load (a) Combination	Nu (b) (kip)	Vu (c) (kip)	φVn (d) (kip)	D/C (e)
Forebay Long Wall (2 layers of #11@6")	Normal	-68	2010	16083	0.13
	Wind	-65	2010	16083	0.13
	SSE	-387	1364	16083	0.08
	Tornado	13	1389	16083	0.09
	PMH	70	1550	16083	0.10
	SPH	-14	2201	16083	0.14
UHS MWIS Water Basin Side Wall (1 layers of #11@9")	Normal	-2295	2279	9921	0.23
	Wind	-2317	2285	9921	0.23
	SSE	-1498	760	5618	0.14
	Tornado	-1481	1550	9921	0.16
	PMH	-1822	1110	9921	0.11
	SPH	-2226	2156	9921	0.22
UHS MWIS Walls	Normal	-958	140	7067	0.02

2' Thick (1 layer of #9@9")	Wind	-337	75	4077	0.02
	SSE	-686	312	4077	0.08
	Tornado	-619	113	7067	0.02
	PMH	-775	267	4077	0.07
	SPH	-731	190	7067	0.03

Notes:

(a) Load combinations are defined in Section 3E.4.3

(b) Nu = Normal force on friction interface (positive for tension)

(c) Vu = Shear demand, vector sum of in-plane and out-of-plane shear

(d) φVn = Nominal shear friction strength

(e) D/C = Demand/Capacity, i.e. Vu/φVn

Figure 3E-2— {Reinforcement for Forebay and UHS Makeup Water Intake Structure Basemat}

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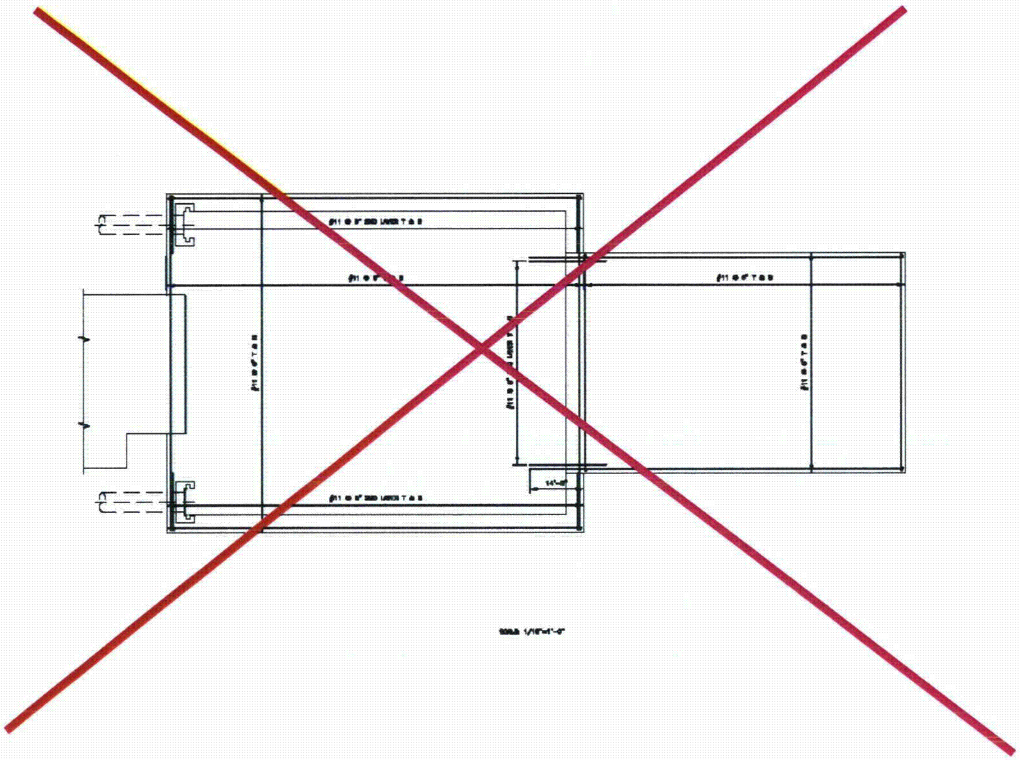

Figure 3E-2— {Reinforcement for Forebay and UHS Makeup Water Intake Structure Basemat}

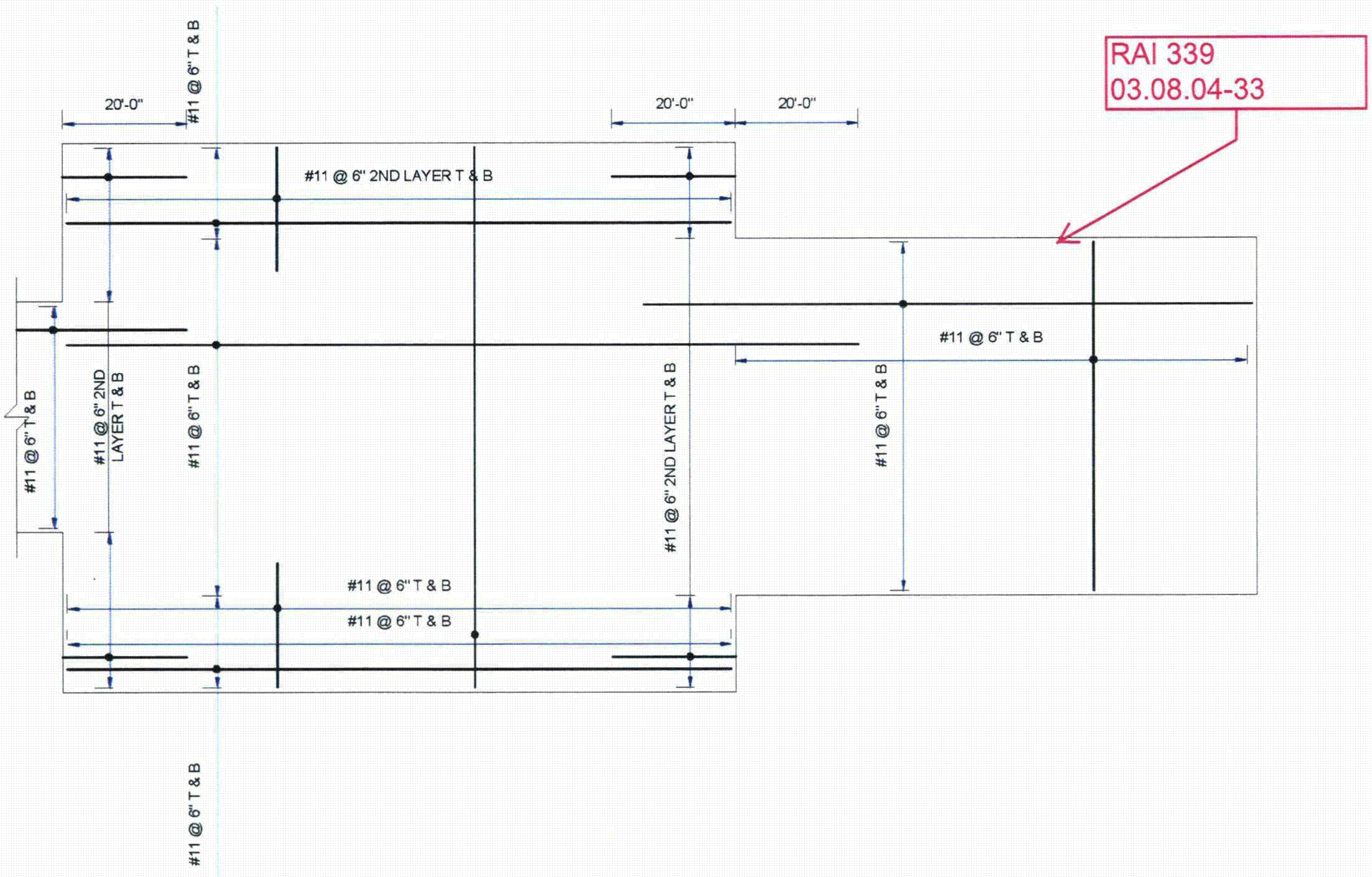

Figure 3E-3— {Reinforcement for Forebay and UHS Makeup Water Intake Structure Walls - UHS Makeup Water Intake Structure Side Wall (Section B)}

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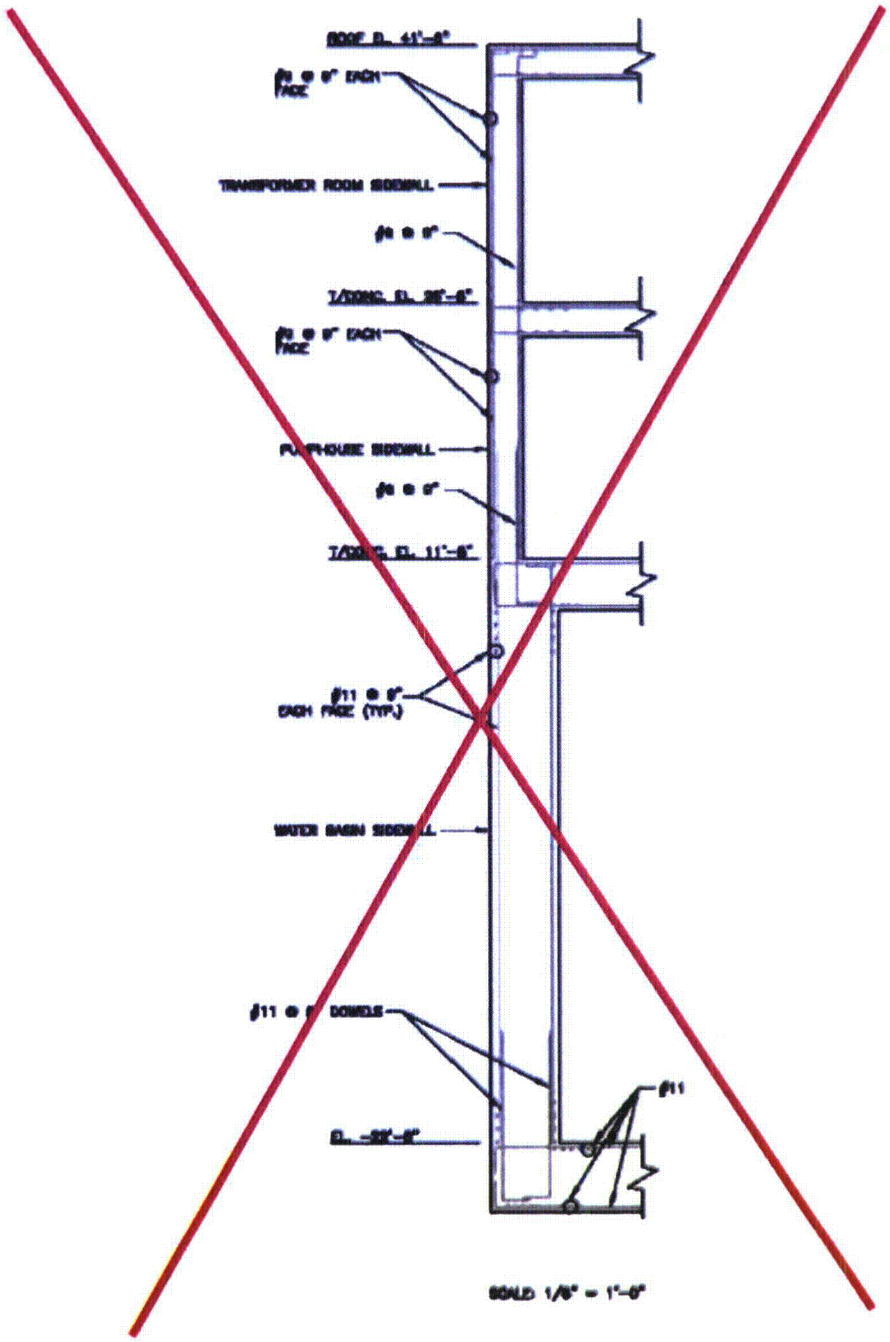

Insert for new Figure 3.E-3

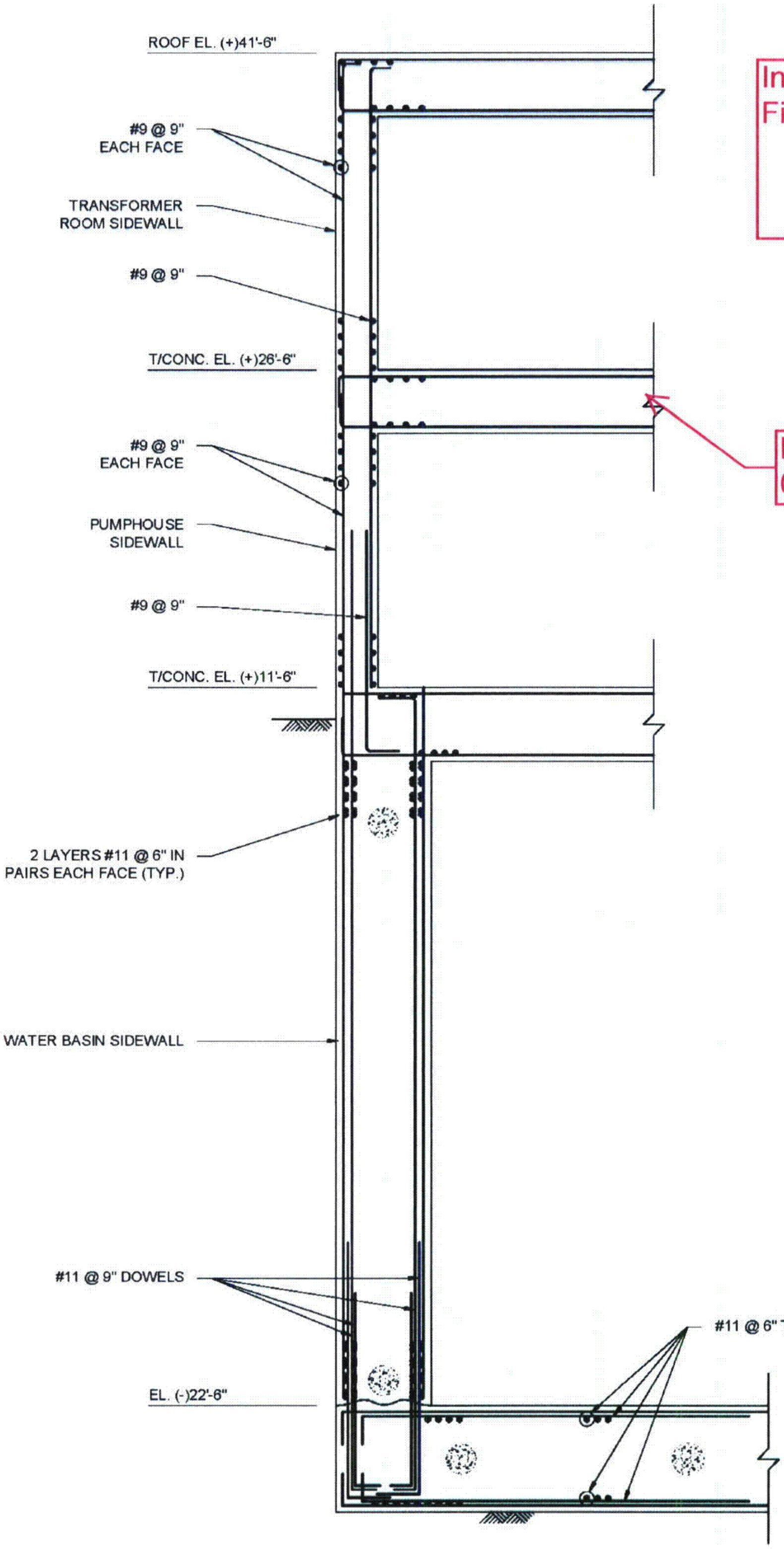

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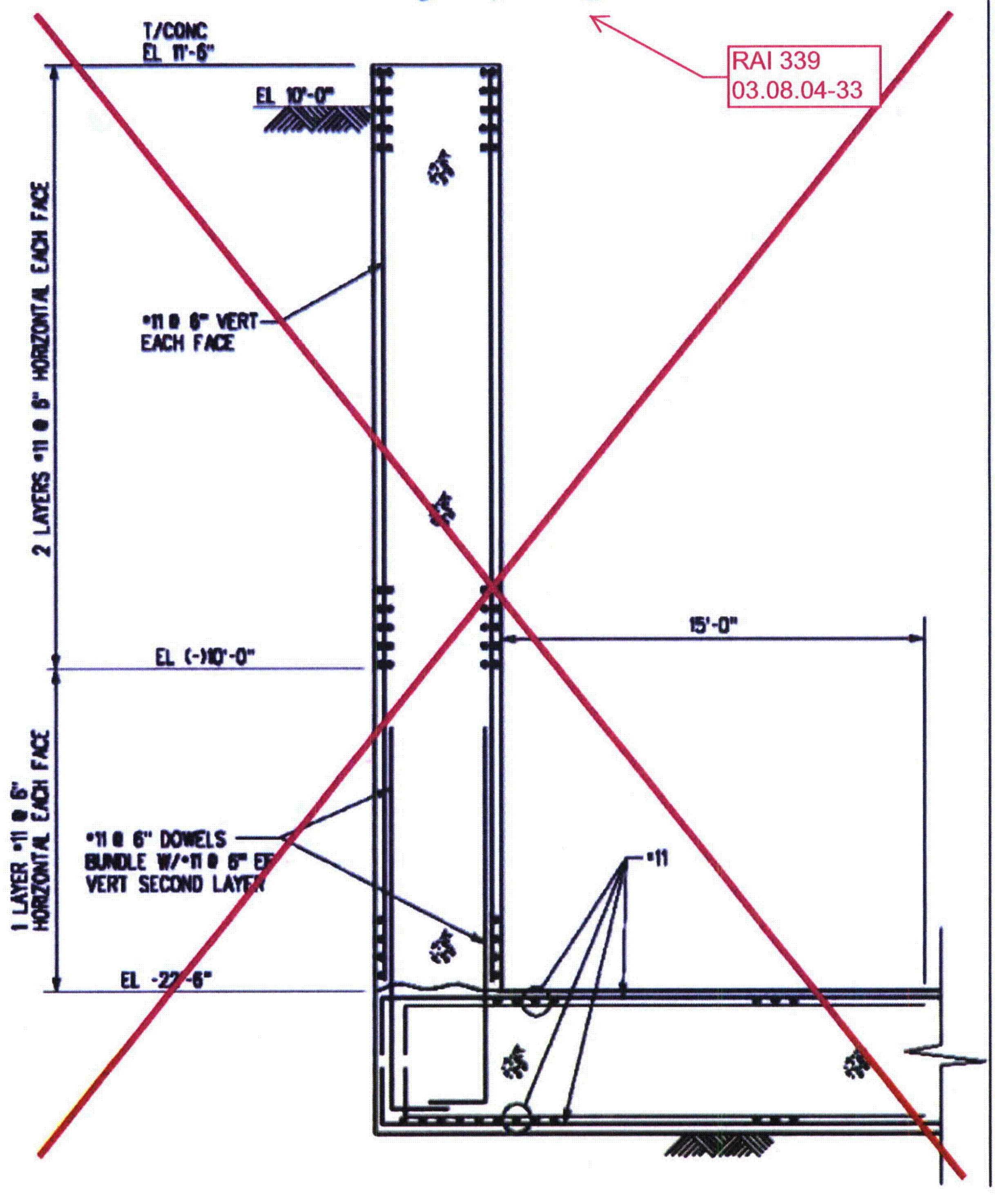
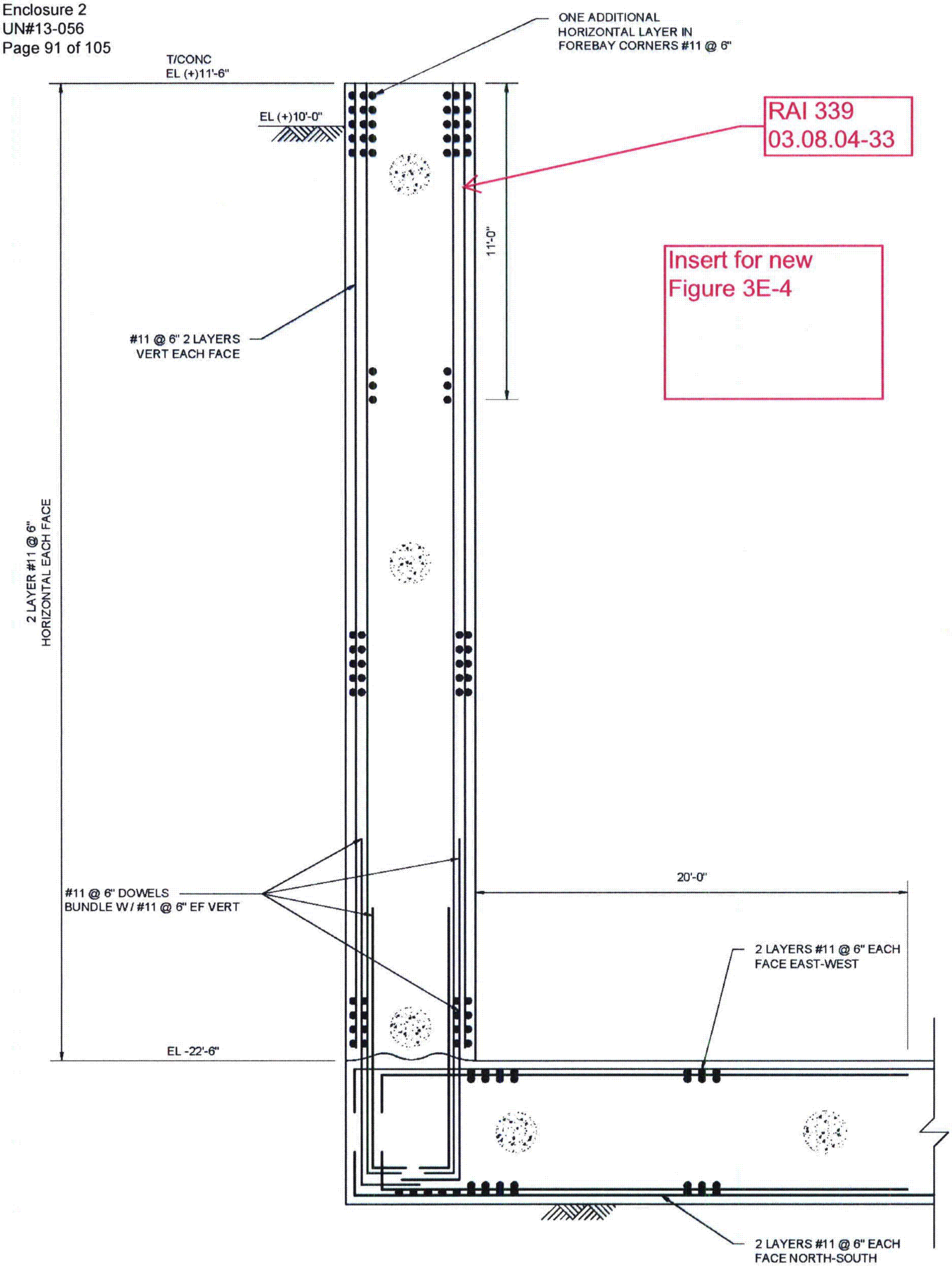
Figure 3E-4— {Reinforcement for Forebay and UHS Makeup Water Intake Structure Walls - Forebay Long Wall (Section C)}




ONE ADDITIONAL
HORIZONTAL LAYER IN
FOREBAY CORNERS #11 @ 6"
T/CONC
EL (+)11'-6"
EL (+)10'-0"
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11'-0"
Insert for new
Figure 3E-4
#11 @ 6" 2 LAYERS
VERT EACH FACE
2 LAYER #11 @ 6"
HORIZONTAL EACH FACE
#11 @ 6" DOWELS
BUNDLE W/ #11 @ 6" EF VERT
20'-0"
2 LAYERS #11 @ 6" EACH
FACE EAST-WEST
EL -22'-6"
2 LAYERS #11 @ 6" EACH
FACE NORTH-SOUTH

Figure 3E-5— {Isometric View of the Common Basemat Intake Structures STAAD Pro Model for Static Analyses}

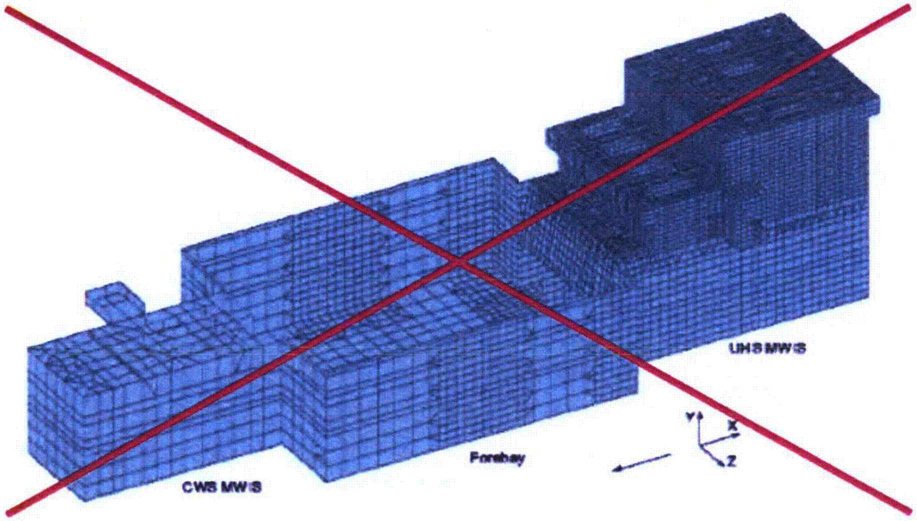

Figure 3E-5— {Isometric View of the Common Basemat Intake Structures STAAD Pro Model for Static Analyses}

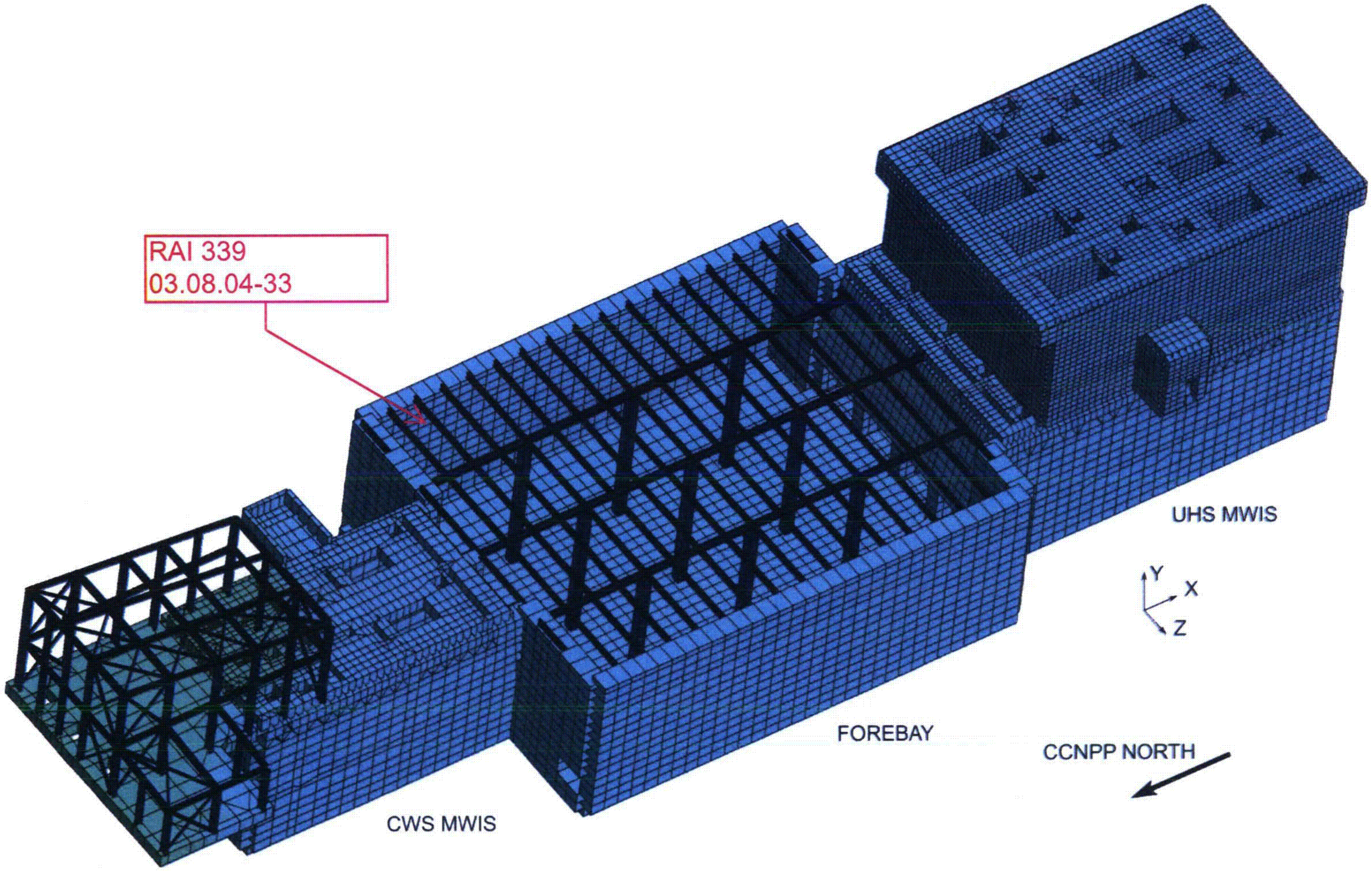

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Table 3F-6— {Best Estimate Site SSE Strain-Compatible Profiles for the Intake Area}

Layer No.	Thickness [ft]	Top Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
1	3.5	0.0	0.145	666.6	1387.6	2.13
2	3.5	3.5	0.145	629.4	1310.1	3.46
3	3.0	7.0	0.145	596.9	3043.7	4.02
4	4.5	10.0	0.145	590.5	3010.8	5.49
5	3.5	14.5	0.145	587.0	2993.0	6.13
6	2.5	18.0	0.145	590.5	3011.2	6.51
7	4.0	20.5	0.145	606.9	3094.7	6.89
8	4.0	24.5	0.145	631.8	3221.5	7.03
9	4.0	28.5	0.145	634.8	3236.6	7.25
10	5.0	32.5	0.145	632.0	3222.4	7.53
11	3.5	37.5	0.115	1118.4	5125.2	2.10
12	4.0	41.0	0.115	1116.4	5116.0	2.13
13	4.0	45.0	0.115	1114.2	5106.0	2.16
14	4.0	49.0	0.115	1112.1	5096.4	2.19
15	5.0	53.0	0.105	1097.1	5027.4	2.01
16	5.0	58.0	0.105	1093.0	5008.8	2.08
17	5.0	63.0	0.105	1090.6	4997.9	2.13
18	7.0	68.0	0.105	1088.0	4985.9	2.19
19	10.0	75.0	0.105	1084.7	4970.7	2.26
20	10.0	85.0	0.105	1081.3	4955.0	2.33
21	10.0	95.0	0.105	1078.2	4941.1	2.39
22	10.0	105.0	0.106	1072.1	4913.1	2.29
23	10.0	115.0	0.113	1031.5	4800.0	1.53
24	8.0	125.0	0.115	1021.3	4800.0	1.39
25	8.0	133.0	0.113	1027.8	4800.0	1.39
26	9.0	141.0	0.107	1053.2	4826.4	1.39
27	10.0	150.0	0.105	1060.2	4858.4	1.38
28	10.0	160.0	0.105	1058.2	4849.4	1.39
29	10.0	170.0	0.105	1056.2	4840.3	1.40
30	10.0	180.0	0.105	1054.3	4831.2	1.41
31	10.0	190.0	0.105	1060.3	4858.8	1.43
32	10.0	200.0	0.108	1133.4	5194.0	1.55
33	10.0	210.0	0.119	1415.0	4800.0	1.90
34	10.0	220.0	0.125	1700.6	5640.3	2.09
35	10.0	230.0	0.125	2049.2	5517.6	1.88
36	10.0	240.0	0.125	2083.2	5944.6	1.90
37	10.0	250.0	0.125	2001.5	5711.4	1.95
38	10.0	260.0	0.125	1992.0	5684.3	1.98
39	10.0	270.0	0.125	1966.5	5611.5	2.04
40	10.0	280.0	0.125	1889.1	5771.4	2.11

Table 3F-6— {Best Estimate Site SSE Strain-Compatible Profiles for the Intake Area}

Layer No.	Thickness [ft]	Top Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
~~41~~	~~10.0~~	~~290.0~~	~~0.125~~	~~1877.8~~	~~5736.7~~	~~2.09~~
~~42~~	~~10.0~~	~~300.0~~	~~0.125~~	~~1876.4~~	~~5732.4~~	~~2.10~~
~~43~~	~~10.0~~	~~310.0~~	~~0.125~~	~~1885.0~~	~~5758.8~~	~~2.09~~
~~44~~	~~7.0~~	~~320.0~~	~~0.123~~	~~1915.2~~	~~5851.0~~	~~2.07~~
~~45~~	~~6.0~~	~~327.0~~	~~0.118~~	~~2034.5~~	~~4983.4~~	~~2.00~~
~~46~~	~~7.0~~	~~333.0~~	~~0.116~~	~~2098.4~~	~~5140.0~~	~~1.99~~
~~47~~	~~10.0~~	~~340.0~~	~~0.115~~	~~2114.0~~	~~5178.2~~	~~2.01~~
~~48~~	~~10.0~~	~~350.0~~	~~0.115~~	~~2113.4~~	~~5176.7~~	~~2.02~~
~~49~~	~~10.0~~	~~360.0~~	~~0.115~~	~~2112.8~~	~~5175.2~~	~~2.03~~
~~50~~	~~10.0~~	~~370.0~~	~~0.115~~	~~2112.2~~	~~5173.8~~	~~2.03~~

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Layer No.	Thickness [ft]	Top-Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
1	3.50	0.00	0.145	660.1	1374.2	2.23
2	3.50	3.50	0.145	613.9	1277.9	3.72
3	3.00	7.00	0.145	571.0	2911.6	5.05
4	4.50	10.00	0.145	554.8	2828.8	6.08
5	3.50	14.50	0.145	540.3	2755.0	6.88
6	2.50	18.00	0.145	539.1	2748.6	7.36
7	4.00	20.50	0.145	547.0	2789.0	7.79
8	4.00	24.50	0.145	562.0	2865.7	8.08
9	4.00	28.50	0.145	561.0	2860.5	8.38
10	5.00	32.50	0.145	552.1	2815.2	8.79
11	3.50	37.50	0.115	1109.2	5083.2	2.22
12	4.00	41.00	0.115	1106.5	5070.7	2.26
13	4.00	45.00	0.115	1103.4	5056.3	2.30
14	4.00	49.00	0.115	1100.3	5042.0	2.34
15	5.00	53.00	0.105	1083.1	4963.6	2.30
16	5.00	58.00	0.105	1078.2	4941.1	2.39
17	5.00	63.00	0.105	1075.2	4927.2	2.46
18	7.00	68.00	0.105	1072.0	4912.4	2.52
19	10.00	75.00	0.105	1067.9	4893.7	2.61
20	10.00	85.00	0.105	1063.7	4874.7	2.69
21	10.00	95.00	0.105	1060.1	4858.1	2.77
22	10.00	105.00	0.106	1053.2	4826.4	2.63
23	10.00	115.00	0.113	1010.4	4800.0	1.65
24	8.00	125.00	0.115	999.1	4800.0	1.47
25	8.00	133.00	0.113	1005.4	4800.0	1.49
26	9.00	141.00	0.107	1030.7	4800.0	1.51
27	10.00	150.00	0.105	1037.5	4800.0	1.50
28	10.00	160.00	0.105	1034.8	4800.0	1.52
29	10.00	170.00	0.105	1032.1	4800.0	1.53
30	10.00	180.00	0.105	1029.2	4800.0	1.55

31	10.00	190.00	0.105	1034.4	4800.0	1.57
32	10.00	200.00	0.108	1107.0	5073.0	1.70
33	10.00	210.00	0.119	1389.7	4800.0	2.09
34	10.00	220.00	0.125	1681.3	5576.3	2.28
35	10.00	230.00	0.125	2033.5	5475.4	2.02
36	10.00	240.00	0.125	2069.7	5906.1	2.05
37	10.00	250.00	0.125	1985.2	5665.0	2.11
38	10.00	260.00	0.125	1974.3	5633.9	2.13
39	10.00	270.00	0.125	1947.2	5556.4	2.22
40	10.00	280.00	0.125	1866.8	5703.2	2.32
41	10.00	290.00	0.125	1854.2	5664.8	2.30
42	10.00	300.00	0.125	1851.7	5656.9	2.31
43	10.00	310.00	0.125	1859.5	5680.8	2.30
44	7.00	320.00	0.123	1889.2	5771.4	2.29
45	6.00	327.00	0.118	2009.9	4923.3	2.20
46	7.00	333.00	0.116	2073.9	5079.9	2.19
47	10.00	340.00	0.115	2089.2	5117.4	2.22
48	10.00	350.00	0.115	2087.5	5113.2	2.23
49	10.00	360.00	0.115	2085.8	5109.2	2.24
50	10.00	370.00	0.115	2084.3	5105.4	2.25

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Table 3F-7— {Lower Bound Site SSE Strain-Compatible Profiles for the Intake Area}

Layer No.	Thickness [ft]	Top Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
1	3.5	0.0	0.145	535.8	1115.4	3.19
2	3.5	3.5	0.145	462.2	962.1	5.78
3	3.0	7.0	0.145	418.3	2132.9	7.96
4	4.5	10.0	0.145	395.2	2014.9	9.44
5	3.5	14.5	0.145	374.3	1908.5	10.57
6	2.5	18.0	0.145	373.1	1902.6	11.12
7	4.0	20.5	0.145	384.5	1960.6	11.53
8	4.0	24.5	0.145	412.0	2100.5	11.52
9	4.0	28.5	0.145	403.9	2059.7	11.88
10	5.0	32.5	0.145	405.0	2065.1	12.19
11	3.5	37.5	0.115	913.2	4656.3	2.62
12	4.0	41.0	0.115	911.5	4648.0	2.67
13	4.0	45.0	0.115	909.7	4638.8	2.72
14	4.0	49.0	0.115	908.0	4630.1	2.77
15	5.0	53.0	0.105	895.8	4567.5	2.66
16	5.0	58.0	0.105	892.4	4550.6	2.79
17	5.0	63.0	0.105	890.5	4540.6	2.85
18	7.0	68.0	0.105	888.4	4529.7	2.92
19	10.0	75.0	0.105	885.6	4515.9	3.01
20	10.0	85.0	0.105	882.9	4501.7	3.11
21	10.0	95.0	0.105	880.4	4489.1	3.19
22	10.0	105.0	0.106	875.4	4463.6	3.16
23	10.0	115.0	0.113	842.2	4294.4	2.22
24	8.0	125.0	0.115	833.9	4251.9	1.76
25	8.0	133.0	0.113	839.2	4279.2	1.75
26	9.0	141.0	0.107	859.9	4384.8	1.79
27	10.0	150.0	0.105	865.6	4414.0	1.78
28	10.0	160.0	0.105	864.0	4405.7	1.79
29	10.0	170.0	0.105	862.4	4397.5	1.80
30	10.0	180.0	0.105	860.8	4389.2	1.82
31	10.0	190.0	0.105	865.7	4414.3	1.86
32	10.0	200.0	0.108	925.4	4718.9	2.14
33	10.0	210.0	0.119	1093.3	4800.0	2.51
34	10.0	220.0	0.125	1388.5	4800.0	2.51
35	10.0	230.0	0.125	1659.2	4800.0	2.28
36	10.0	240.0	0.125	1700.9	4853.7	2.22
37	10.0	250.0	0.125	1634.2	4800.0	2.30
38	10.0	260.0	0.125	1626.5	4800.0	2.37
39	10.0	270.0	0.125	1605.6	4800.0	2.44
40	10.0	280.0	0.125	1542.5	4800.0	2.49

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Table 3F-7— {Lower Bound Site SSE Strain-Compatible Profiles for the Intake Area}

Layer No.	Thickness [ft]	Top Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
41	10.0	290.0	0.125	1533.2	4800.0	2.46
42	10.0	300.0	0.125	1532.0	4800.0	2.48
43	10.0	310.0	0.125	1539.1	4800.0	2.46
44	7.0	320.0	0.123	1563.7	4800.0	2.43
45	6.0	327.0	0.118	1661.1	4800.0	2.35
46	7.0	333.0	0.116	1713.3	4800.0	2.33
47	10.0	340.0	0.115	1726.1	4800.0	2.32
48	10.0	350.0	0.115	1725.6	4800.0	2.33
49	10.0	360.0	0.115	1725.1	4800.0	2.35
50	10.0	370.0	0.115	1724.6	4800.0	2.35

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Layer No.	Thickness [ft]	Top-Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
1	3.50	0.00	0.145	527.3	1097.7	3.38
2	3.50	3.50	0.145	440.8	917.7	6.34
3	3.00	7.00	0.145	382.2	1948.8	8.91
4	4.50	10.00	0.145	350.7	1788.2	10.63
5	3.50	14.50	0.145	320.4	1633.8	11.98
6	2.50	18.00	0.145	316.5	1613.8	12.65
7	4.00	20.50	0.145	318.8	1625.5	13.23
8	4.00	24.50	0.145	334.1	1703.4	13.44
9	4.00	28.50	0.145	327.2	1668.3	13.87
10	5.00	32.50	0.145	321.7	1640.2	14.23
11	3.50	37.50	0.115	905.7	4618.2	2.81
12	4.00	41.00	0.115	903.5	4606.8	2.88
13	4.00	45.00	0.115	900.9	4593.7	2.94
14	4.00	49.00	0.115	898.4	4580.7	3.00
15	5.00	53.00	0.105	884.4	4509.5	3.09
16	5.00	58.00	0.105	880.4	4489.0	3.22
17	5.00	63.00	0.105	877.9	4476.5	3.30
18	7.00	68.00	0.105	875.3	4463.0	3.39
19	10.00	75.00	0.105	871.9	4446.0	3.49
20	10.00	85.00	0.105	868.5	4428.7	3.60
21	10.00	95.00	0.105	865.6	4413.6	3.69
22	10.00	105.00	0.106	859.9	4384.8	3.69
23	10.00	115.00	0.113	824.9	4206.4	2.47
24	8.00	125.00	0.115	815.7	4159.4	1.91
25	8.00	133.00	0.113	820.9	4185.9	1.92
26	9.00	141.00	0.107	841.6	4291.4	1.98
27	10.00	150.00	0.105	847.1	4319.4	1.97
28	10.00	160.00	0.105	844.9	4308.2	2.01
29	10.00	170.00	0.105	842.7	4296.8	2.03
30	10.00	180.00	0.105	840.3	4285.0	2.07

31	10.00	190.00	0.105	844.6	4306.6	2.12
32	10.00	200.00	0.108	897.6	4576.8	2.42
33	10.00	210.00	0.119	1058.7	4800.0	2.81
34	10.00	220.00	0.125	1372.8	4800.0	2.81
35	10.00	230.00	0.125	1636.6	4800.0	2.50
36	10.00	240.00	0.125	1689.9	4822.4	2.42
37	10.00	250.00	0.125	1620.9	4800.0	2.56
38	10.00	260.00	0.125	1612.0	4800.0	2.61
39	10.00	270.00	0.125	1589.9	4800.0	2.73
40	10.00	280.00	0.125	1524.2	4800.0	2.83
41	10.00	290.00	0.125	1514.0	4800.0	2.80
42	10.00	300.00	0.125	1511.9	4800.0	2.85
43	10.00	310.00	0.125	1518.2	4800.0	2.81
44	7.00	320.00	0.123	1542.5	4800.0	2.79
45	6.00	327.00	0.118	1641.1	4800.0	2.67
46	7.00	333.00	0.116	1693.3	4800.0	2.63
47	10.00	340.00	0.115	1705.8	4800.0	2.65
48	10.00	350.00	0.115	1704.4	4800.0	2.66
49	10.00	360.00	0.115	1703.1	4800.0	2.67
50	10.00	370.00	0.115	1701.8	4800.0	2.69

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Table 3F-8— {Upper Bound Site SSE Strain-Compatible Profiles for the Intake Area

Layer No.	Thickness [ft]	Top Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
1	3.5	0.0	0.145	829.2	1726.1	1.42
2	3.5	3.5	0.145	857.0	1784.0	2.0[illegible]
3	3.0	7.0	0.145	851.8	4343.4	[illegible].68
4	4.5	10.0	0.145	882.3	4498.9	3.19
5	3.5	14.5	0.145	920.5	4693.8	3.56
6	2.5	18.0	0.145	934.6	4765.8	3.81
7	4.0	20.5	0.145	958.0	4800.0	4.12
8	4.0	24.5	0.145	968.9	4800.0	4.29
9	4.0	28.5	0.145	997.5	4800.0	4.43
10	5.0	32.5	0.145	986.1	4800.0	4.65
11	3.5	37.5	0.115	1369.8	6277.0	1.68
12	4.0	41.0	0.115	1367.3	6265.8	1.70
13	4.0	45.0	0.115	1364.6	6253.5	1.72
14	4.0	49.0	0.115	1362.1	6241.8	1.73
15	5.0	53.0	0.105	1343.6	6157.3	1.52
16	5.0	58.0	0.105	1338.7	6134.5	1.55
17	5.0	63.0	0.105	1335.7	6121.1	1.59
18	7.0	68.0	0.105	1332.5	6106.4	1.64
19	10.0	75.0	0.105	1328.5	6087.8	1.70
20	10.0	85.0	0.105	1324.3	6068.6	1.75
21	10.0	95.0	0.105	1320.6	6051.6	1.79
22	10.0	105.0	0.106	1313.1	6017.3	1.66
23	10.0	115.0	0.113	1263.3	5789.2	1.05
24	8.0	125.0	0.115	1250.8	5731.8	1.10
25	8.0	133.0	0.113	1258.8	5768.7	1.11
26	9.0	141.0	0.107	1289.9	5911.1	1.08
27	10.0	150.0	0.105	1298.5	5950.3	1.07
28	10.0	160.0	0.105	1296.1	5939.3	1.08
29	10.0	170.0	0.105	1293.6	5928.2	1.09
30	10.0	180.0	0.105	1291.2	5917.0	1.09
31	10.0	190.0	0.105	1298.6	5950.8	1.10
32	10.0	200.0	0.108	1388.2	6361.4	1.12
33	10.0	210.0	0.119	1831.2	6[illegible]73.3	1.44
34	10.0	220.0	0.125	2082.8	690[illegible].9	1.74
35	10.0	230.0	0.125	2530.8	6814.4	1.55
36	10.0	240.0	0.125	2551.4	7280.6	1.63
37	10.0	250.0	0.125	2451.3	6995.0	1.65
38	10.0	260.0	0.125	2439.7	6961.8	1.66
39	10.0	270.0	0.125	2408.4	6872.7	1.71
40	10.0	280.0	0.125	2313.7	7068.5	1.7[illegible]

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Table 3F-8— {Upper Bound Site SSE Strain-Compatible Profiles for the Intake Area

Layer No.	Thickness [ft]	Top Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
41	10.0	290.0	0.125	2299.8	7026.0	1.77
42	10.0	300.0	0.125	2298.1	7020.7	1.78
43	10.0	310.0	0.125	2308.7	7053.1	1.78
44	7.0	320.0	0.123	2345.6	7165.9	1.76
45	6.0	327.0	0.118	2491.7	6103.4	1.70
46	7.0	333.0	0.116	2570.0	6295.2	1.70
47	10.0	340.0	0.115	2589.1	6342.0	1.74
48	10.0	350.0	0.115	2588.4	6340.1	1.75
49	10.0	360.0	0.115	2587.6	6338.3	1.75
50	10.0	370.0	0.115	2586.9	6336.5	1.75

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Layer No.	Thickness [ft]	Top-Depth [ft]	Unit Weight [kcf]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damping [%]
1	3.50	0.00	0.145	826.4	1720.2	1.47
2	3.50	3.50	0.145	854.8	1779.4	2.18
3	3.00	7.00	0.145	853.1	4350.2	2.86
4	4.50	10.00	0.145	877.6	4474.9	3.48
5	3.50	14.50	0.145	911.1	4645.6	3.95
6	2.50	18.00	0.145	918.1	4681.5	4.28
7	4.00	20.50	0.145	938.5	4785.3	4.59
8	4.00	24.50	0.145	945.5	4800.0	4.86
9	4.00	28.50	0.145	961.9	4800.0	5.06
10	5.00	32.50	0.145	947.6	4800.0	5.43
11	3.50	37.50	0.115	1358.5	6225.6	1.75
12	4.00	41.00	0.115	1355.2	6210.3	1.77
13	4.00	45.00	0.115	1351.4	6192.7	1.80
14	4.00	49.00	0.115	1347.5	6175.1	1.83
15	5.00	53.00	0.105	1326.6	6079.1	1.71
16	5.00	58.00	0.105	1320.6	6051.5	1.78
17	5.00	63.00	0.105	1316.9	6034.6	1.83
18	7.00	68.00	0.105	1312.9	6016.4	1.87
19	10.00	75.00	0.105	1307.9	5993.6	1.95
20	10.00	85.00	0.105	1302.8	5970.2	2.01
21	10.00	95.00	0.105	1298.4	5949.9	2.08
22	10.00	105.00	0.106	1289.9	5911.1	1.88
23	10.00	115.00	0.113	1237.4	5670.6	1.10
24	8.00	125.00	0.115	1223.6	5607.2	1.13
25	8.00	133.00	0.113	1231.4	5643.0	1.16
26	9.00	141.00	0.107	1262.4	5785.1	1.15
27	10.00	150.00	0.105	1270.7	5822.9	1.14
28	10.00	160.00	0.105	1267.4	5807.8	1.15
29	10.00	170.00	0.105	1264.0	5792.5	1.15
30	10.00	180.00	0.105	1260.5	5776.4	1.16

31	10.00	190.00	0.105	1266.9	5805.7	1.16
32	10.00	200.00	0.108	1365.3	6256.8	1.20
33	10.00	210.00	0.119	1824.2	6050.2	1.55
34	10.00	220.00	0.125	2059.2	6829.5	1.85
35	10.00	230.00	0.125	2526.7	6803.4	1.63
36	10.00	240.00	0.125	2534.9	7233.5	1.74
37	10.00	250.00	0.125	2431.4	6938.2	1.74
38	10.00	260.00	0.125	2418.0	6900.0	1.74
39	10.00	270.00	0.125	2384.8	6805.2	1.80
40	10.00	280.00	0.125	2286.4	6985.0	1.90
41	10.00	290.00	0.125	2271.0	6938.0	1.89
42	10.00	300.00	0.125	2267.8	6928.3	1.88
43	10.00	310.00	0.125	2277.4	6957.5	1.89
44	7.00	320.00	0.123	2313.7	7068.6	1.88
45	6.00	327.00	0.118	2461.6	6029.8	1.81
46	7.00	333.00	0.116	2540.0	6221.6	1.82
47	10.00	340.00	0.115	2558.7	6267.5	1.86
48	10.00	350.00	0.115	2556.6	6262.4	1.87
49	10.00	360.00	0.115	2554.6	6257.5	1.88
50	10.00	370.00	0.115	2552.7	6252.9	1.88

Figure 3F-32— {Shear Wave Velocity Profiles Strain-Compatible with Site SSE for the Intake Area}

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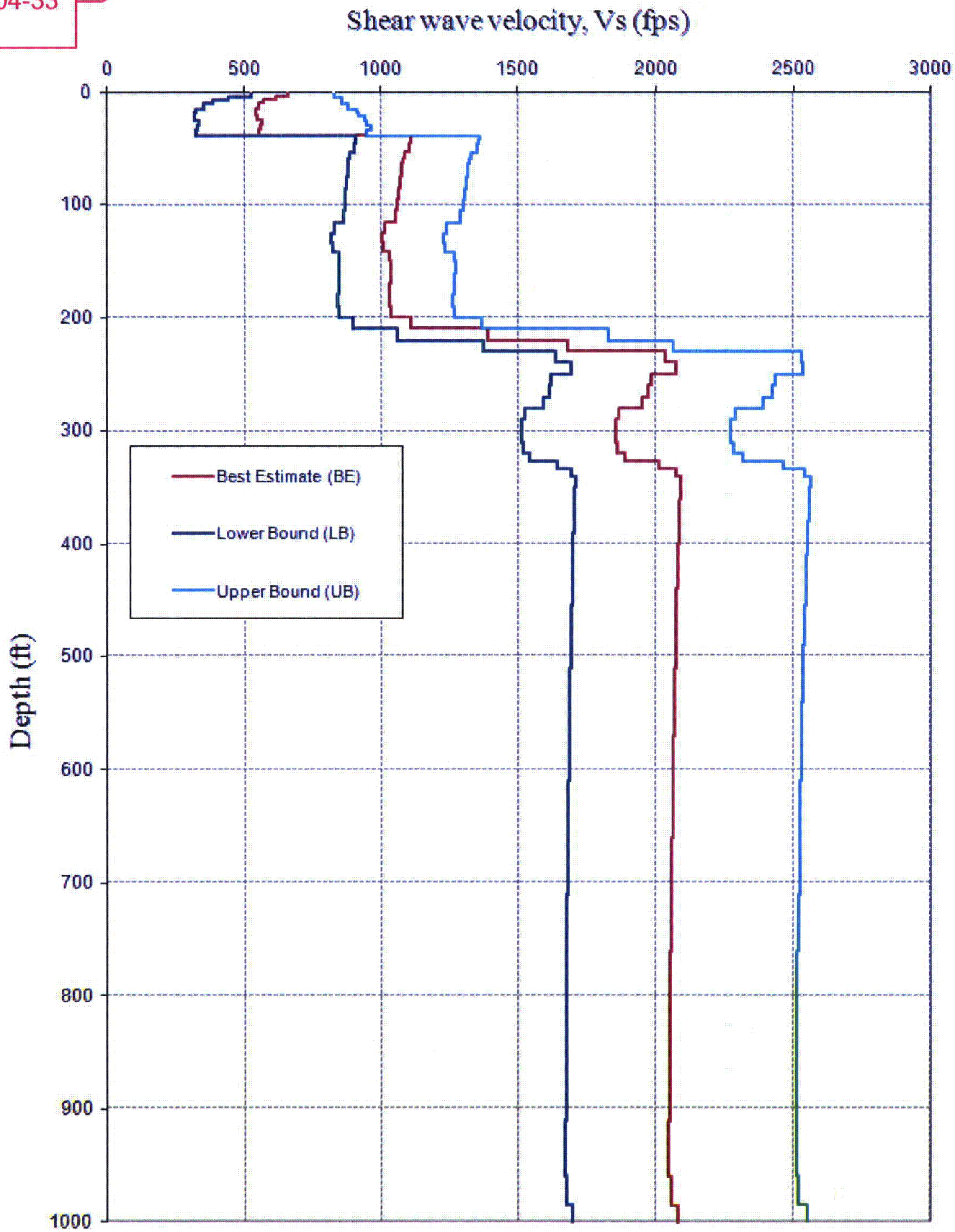

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Figure 3F-33— {Damping Profiles Strain-Compatible with Site SSE for the Intake Area}

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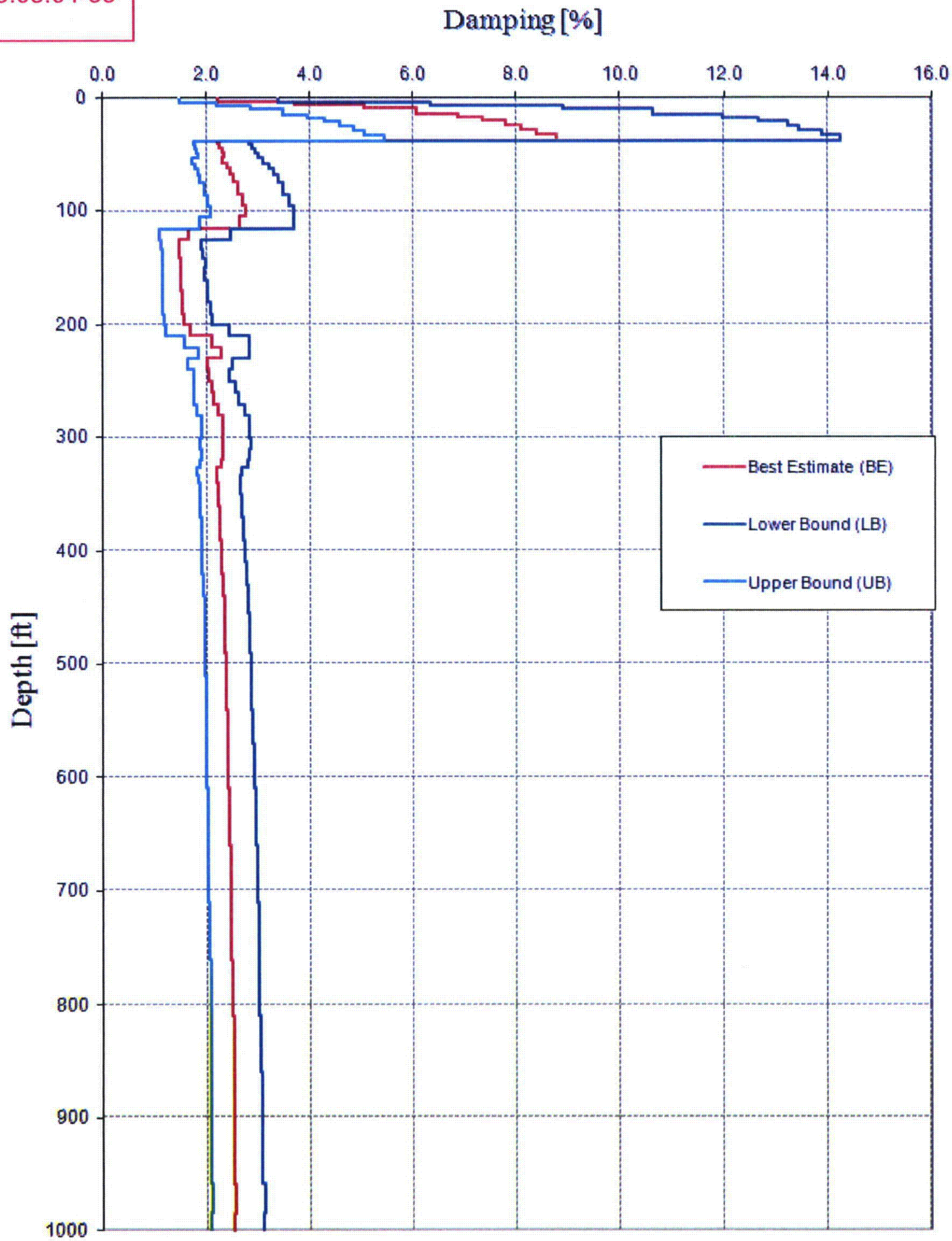

Figure 3F-34— {Site SSE P-Wave Velocity Profiles for the Intake Area}

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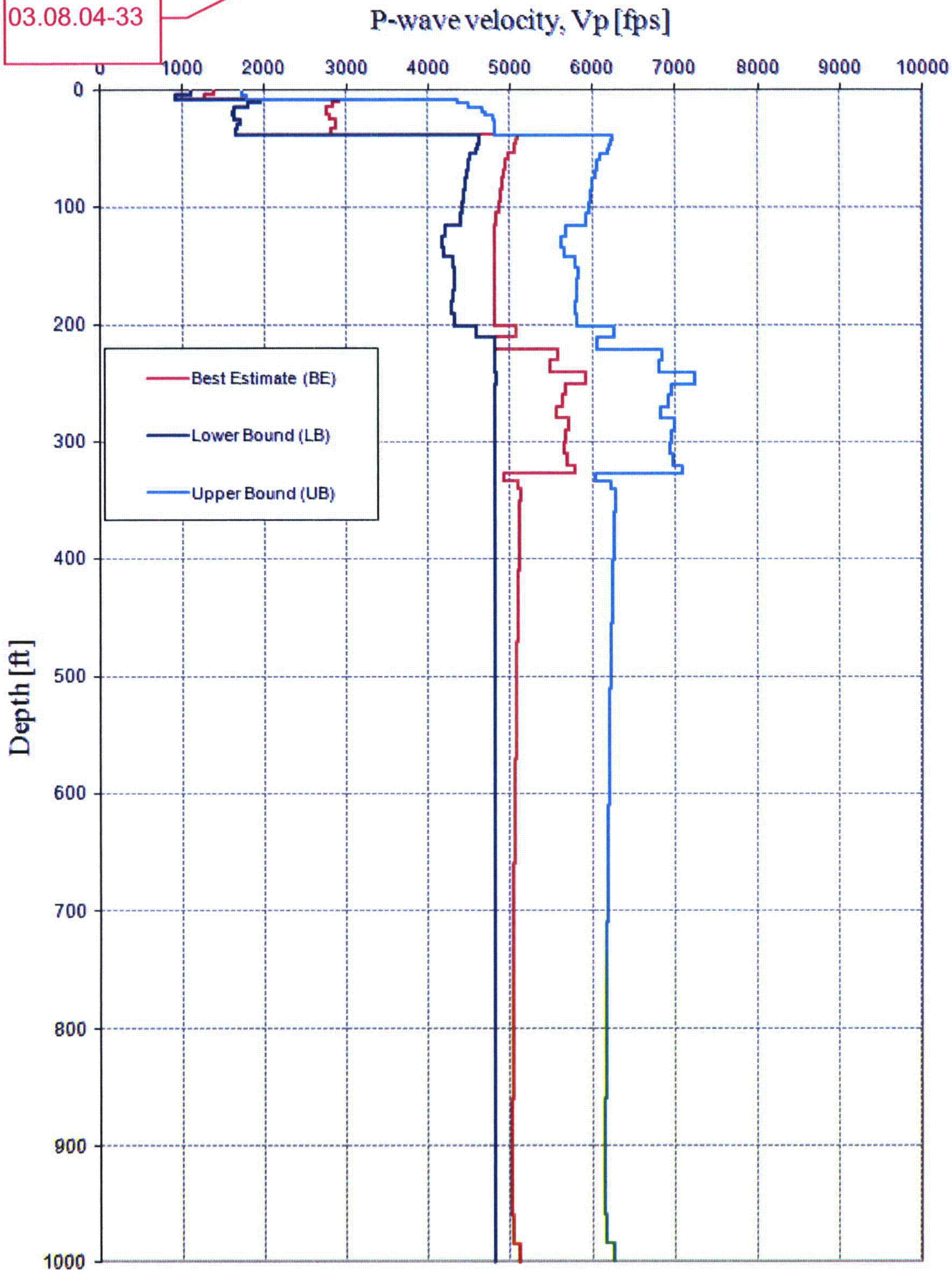

Enclosure 3

**Table of Changes to CCNPP Unit 3 COLA
Associated with the Response to
RAI 315, Question 03.07.02-64 (Part C),
Calvert Cliffs Nuclear Power Plant, Unit 3**

Table of Changes to CCNPP Unit 3 COLA

Associated with the Response to RAI No. 315

Change ID #	Subsection	Type of Change	Description of Change
Part 2 – FSAR			
CC3-12-0241	3.7.2.2.3	Incorporate COLA markups associated with the response to RAI 330, Question 09.02.05-20[2].	The response to RAI 330, Question 09.02.05-20 involves updating the UHS Makeup Water traveling screen classification to Safety-Related and Seismic Category I in the applicable CCNPP Unit 3 Part 2, FSAR sections and Part 10, ITAAC Tables.
CC3-13-0019	3.7.2.3.2	Incorporate COLA markups associated with the response to RAI 304, Question 03.07.02-56[3].	The response to RAI 304, Question 03.07.02-56 includes a change in the third to last paragraph in Section 3.7.2.3.2 involving normal water level corresponding to MSL. The second to last paragraph in Section 3.7.2.3.2 is also revised to provide new maximum sloshing heights for the UHS Makeup Water Intake Structure and the Forebay.
CC3-10-0302	3.8.4.1.11	Incorporate COLA markups associated with the response to RAI 253, Questions 03.07.02-42, 43, 44, 47, 48, 52, and 53[4].	The second bullet was modified and the third bullet was added as part of the response to RAI 253, Questions 03.07.02-42, 43, 44, 47, 48, 52, and 53.

[2]UniStar Nuclear Energy Letter UN#12-153, from Mark T. Finley to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 330, Ultimate Heat Sink, dated December 20, 2012.

[3]UniStar Nuclear Energy Letter UN#13-008, from Mark T. Finley to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 304, Seismic System Analysis, dated January 23, 2013.

[4]UniStar Nuclear Energy Letter UN#10-285, from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 253, Seismic System Analysis, dated November 16, 2010.

Change ID #	Subsection	Type of Change	Description of Change
CC3-12-0241	3.8.4.1.11	Incorporate COLA markups associated with the response to RAI 330, Question 09.02.05-20[2].	The response to RAI 330, Question 09.02.05-20 involves updating the UHS Makeup Water traveling screen classification to Safety-Related and Seismic Category I in the applicable CCNPP Unit 3 Part 2, FSAR sections and Part 10, ITAAC Tables.
CC3-13-0082	3.7.1.3.3, 3.7.2.1.3, 3.7.2.2.3, 3.7.2.3.2, 3.7.2.4.2.3, 3.7.2.4.3.3, 3.7.2.4.4.3, 3.7.2.4.5.3, 3.7.2.4.6.3, 3.7.2.4.7, 3.7.2.6, 3.7.2.14.3, 3.7.2.16, Table 3.7-5, Table 3.7-6, Table 3.7-7, Table 3.7-10, Figure 3.7-16, Figure 3.7-17, Figure 3.7-18, Figure 3.7-22, Figure 3.7-23, Figure 3.7-24, Figures 3.7-73 through 3.7-81, 3.8.4.1.11, 3.8.4.4.7, 3.8.5.4.6, 3.8.5.5.4, Table 3.8-2, Table 3.8-3, Figure 3.8-5, Table 3E-1, Table 3E-2, Table 3E-3, Table 3E-4, Figure 3E-1, Figure 3E-2, Figure 3E-3, Figure 3E-4, Figure 3E-5, Figure 3F-6, Figure 3F-7, Figure 3F-8, Figure 3F-32, Figure 3F-33, Figure 3F-34	Incorporate COLA markups associated with the response to RAI 315, Question 3.07.02-64 (this response), the RAI 339 Questions 03.08.04-33 and -34 response[1], and the RAI 343 Questions 03.07.02-71 through -74 response[5].	Text, Figure, and Table changes in Sections 3.7 and 3.8 required as part of the response to RAI 315, Question 3.07.02-64 (this response) the RAI 339 Questions 03.08.04-33 and -34 response[1], and the RAI 343 Questions 03.07.02-71 through -74 response[5].

[5]UniStar Nuclear Energy Letter UN#13-058, from Mark T. Finley to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 343, Seismic System Analyses, dated April 30, 2013.