FPL

L-2015-209
10 CFR 52.3

October 29, 2015

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, D.C. 20555-0001

Re: Florida Power & Light Company
Proposed Turkey Point Units 6 and 7
Docket Nos. 52-040 and 52-041
Combined Information from Response to RAI 02.05.04-26 (eRAI 7811)
and FSAR Subsection 2.5.4 Grout Test Program Description

References:

1. FPL Letter L-2015-047 to NRC dated February 19, 2015, FSAR Subsection 2.5.4 Grout Test Program Description
2. FPL Letter L-2015-199 to NRC dated July 15, 2015, Response and Revised Response to NRC Request for Additional Information Letter No. 082 (eRAI 7811) SRP Section 02.05.04 – Stability of Subsurface Materials and Foundations

In Reference 1, Florida Power & Light Company (FPL) provided information on the Grout Test Program for Turkey Point Units 6 & 7. In Reference 2, Attachment 1, FPL provided its response to the Nuclear Regulatory Commission's (NRC) request for additional information (RAI) 02.05.04-26.

During a public meeting on July 23, 2015, the NRC staff requested FPL provide additional detail in the Grout Test Program. The NRC staff also requested FPL to provide one letter that combined the response to RAI 02.05.04-26 and the Grout Test Program for ease of review.

As a result of the public meeting, FPL is providing, as an attachment to this letter, the combined information for the response to NRC RAI 02.05.04-26 and the Grout Test Program with the additional information requested by the NRC staff. The information in this attachment augments and clarifies the information provided in Reference 1 and Reference 2, Attachment 1.

The attachment identifies changes that will be made in a future revision of the Turkey Point Units 6 and 7 Combined License Application (if applicable).

If you have any questions, or need additional information, please contact me at 561-904-3794.

D097
NRO

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

Executed on October 29, 2015

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

WDM/RFB

Attachment: Combined Information from Response to RAI 02.05.04-26 (eRAI 7811) and FSAR Subsection 2.5.4 Grout Test Program Description

cc:
PTN 6 & 7 Project Manager, AP1000 Projects Branch 1, USNRC DNRL/NRO
Regional Administrator, Region II, USNRC
Senior Resident Inspector, USNRC, Turkey Point Plant 3 & 4

INTRODUCTION:

The information in this attachment combines the information on the FSAR Subsection 2.5.4 Grout Test Program provided to the NRC in FPL Letter L-2015-047 dated February 19, 2015 (Reference 1) and from FPL's response to NRC RAI 02.05.04-26 provided in FPL Letter L-2015-199 to NRC dated July 15, 2015 (Reference 2). It should be noted that the information provided in FPL Letter L-2015-047 has been updated based on NRC staff comments provided in a public meeting on July 23, 2015. The information provided below provides additional detail on a Grout Test Program that FPL will perform to assure that the grout plug will perform as expected. The information provided augments and clarifies the information provided in the above referenced letters for the responses for RAI 02.05.04-26 and the FSAR Subsection 2.5.4 Grout Test Program. The changes in the Associated COLA Revisions of this attachment are based on information in COL Application Revision7.

NRC RAI Letter No. PTN-RAI-LTR-082

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-26 (eRAI 7811)

In FSAR Subsection 2.5.4.4.5.5, the applicant commits to performing microgravity surveys on the excavation surfaces of the proposed Units 6 and 7 to detect the presence, or verify the absence, of potential water-filled dissolution features (or voids) beneath the power block. Interpretations of the existing gravity survey anomalies discussed in FSAR 2.5.4.4.5.1 include uncertainties related to significant lateral variations in the shallow soil layers that will be removed prior to construction of Units 6 and 7. FSAR Subsection 2.5.4.4.5.5 states that "The [new] micro- gravity survey will be designed to detect 25-foot diameter spherical voids and cylindrical voids as small as 12 feet in diameter at the base of the 25-foot-thick grout plug at an elevation of approximately –60 feet NAVD 88." If the new gravity data (collected after the excavations are completed) indicate that there might be some gravity anomalies of concern, the applicant plans to drill those areas where such anomalies are observed to understand the source of these anomalies. In accordance with 10 CFR 100.23, please provide the following to assist NRC staff's further review in this area:

1. Clarification and demonstration that only the voids greater than the 25-foot spherical diameter and/or 12-foot cylindrical diameter at an elevation of about –60 feet are critical to the stability of subsurface materials and the integrity of structures, systems, and components (SSCs).
2. If voids smaller than the above mentioned dimensions impact the stability of subsurface materials and are critical to the integrity of SSCs, please provide further information on void size, location and depth in the limestone layers that need to be considered.
3. Additional details on the planned microgravity gravity survey specifically:

a. Address the size of the areas to be investigated

b. Will measurements be conducted along profiles or the entire excavated surfaces surveyed

c. Measurement intervals, and data reduction and processing methodologies to be employed considering the three-dimensionality of the excavations and their impact on the gravity measurements

4. A description of the type of inspection and test program to be followed to reasonably ensure that gravity anomalies resulting from potential underground voids (both within the grouted zones and deeper levels) that are critical to the stability of subsurface materials and the integrity of SSCs are to be appropriately detected, investigated, evaluated and, if necessary, remediated

FPL RESPONSE:

Summary of Conclusions on Karstic Structures from Geological Perspective:

Before describing the method used to determine void size and the numerical models for stability, it is important to re-emphasize the geological conclusion (FSAR Appendix 2.5AA, summarized below) that large voids and karst features are considered unlikely to be present at the site. The geological evaluation of the site is necessary to confirm if karstic structures are present on site and, if so, to establish the likely size and extent of the karstic structures. In this regard, the following are key points to be considered as outlined in FSAR Appendix 2.5AA and the Response to RAI 02.05.01-37.

- Neither the vegetated depressions nor the zones of secondary porosity are considered to pose a hazard of sinkhole development. The vegetated depressions are surficial solution features formed by a subaerial, epigenic process of dissolution caused by downward seepage of slightly acidic meteoric groundwater. The zones of secondary porosity are microkarst features formed in the subsurface by solution enlargement of touching-vug and moldic porosity within paleomixing zones of fresh groundwater and saltwater. An upper zone of secondary porosity has formed in a zone of touching-vug porosity near the contact of the Miami Limestone and the Key Largo Limestone. A lower zone of secondary porosity has formed in a zone of moldic porosity in the underlying Fort Thompson Formation. Microkarst features are in the order of a few centimeters.

- The process that formed the vegetated depressions at the site and its vicinity is ongoing. However, the stratigraphic interval in which they occur will be completely removed during excavation of the nuclear islands. As discussed below, the structure contour and isopach maps indicate that the surficial vegetated depressions do not persist with depth. In addition, the freshwater/saltwater interface is approximately 6 miles inland from the site, and mean sea level rise trend is low (0.78 foot in 100 years); thus, the carbonate dissolution in a fresh groundwater/saltwater mixing zone by the process of shoreline flow is not likely to develop large underground voids.

- As discussed in the Response to RAI 02.05.01-37, available information related to caves, cover collapse sinkholes, springs, submarine sinkholes, paleo-karst collapses, and sag structures in the site vicinity (and in other areas in southeast Florida) suggests that, while dissolution features are present, most are not currently active. Active dissolution is probably limited at Turkey Point Units 6 & 7, as is the potential for deformation due to collapses within existing (i.e., "paleo") dissolution features. Active dissolution associated with karst conduits at the site, as evident in past submarine groundwater discharges, is also likely to be insignificant. Furthermore, the observed collapse structures at Jewfish Creek/Lake Surprise (17 miles from Turkey Point Units 6 & 7), for example, appears to have occurred in the Pleistocene (coincident with sea level lowstands) and thus is not a particularly relevant analog for potentially active (or possible future) surface collapse at (or near) the site. No structural damage or differential settlement has been reported on the bridge that was constructed on the Jewfish Creek/Lake Surprise site that was described in the Response to RAI 02.05.01-37. Although substantial in scale and extent, the seismic sag structures described by Cunningham and Walker (FSAR Reference 2.5.1-958) similarly provide no evidence for post-Pliocene deformation. It seems likely then that comparable collapses in similar features, if present below Turkey Point Units 6 & 7, have already occurred (and are now stabilized).

- Finally, structure contour and isopach maps for the Key Largo Limestone and Fort Thompson Formation and cross-sections prepared with data from the site geotechnical subsurface investigation do not suggest the existence of large underground caverns or sinkholes. Specifically, the following conclusions are obtained (as provided in the Response to RAI 02.05.04-01) for the sizes of potential voids and/or voids filled with soft sediments:

  "...The evaluation of all data (MACTEC, Reference 6; RIZZO, Reference 3) indicate that outside the vegetated depressions and drainages (in vertical borings), a total of 20.1 feet of interpreted tool drops (due to voids and/or voids filled with soft sediments) are observed, in a total of 7918.4 feet cored, for a 0.3 percent of the total cored in 93 borings. Individual drops in the vertical borings range from 0.4 feet to 4 feet (1.5 feet max within the Unit 6 & 7 building footprints). Results from the site investigations (MACTEC, Reference 6; RIZZO, Reference 3), show that interpreted tool drops are found more often under the vegetated depressions and drainages. In the three inclined borings, a total of 15.2 feet of tool drops are observed, in a total of 356.4 feet cored, for a 4.3 percent of the total cored length. Individual drops in the inclined borings range from 0.3 feet to 2.5 feet. Boring locations with interpreted tool drops, among all sampling locations, are shown in Figure 1.

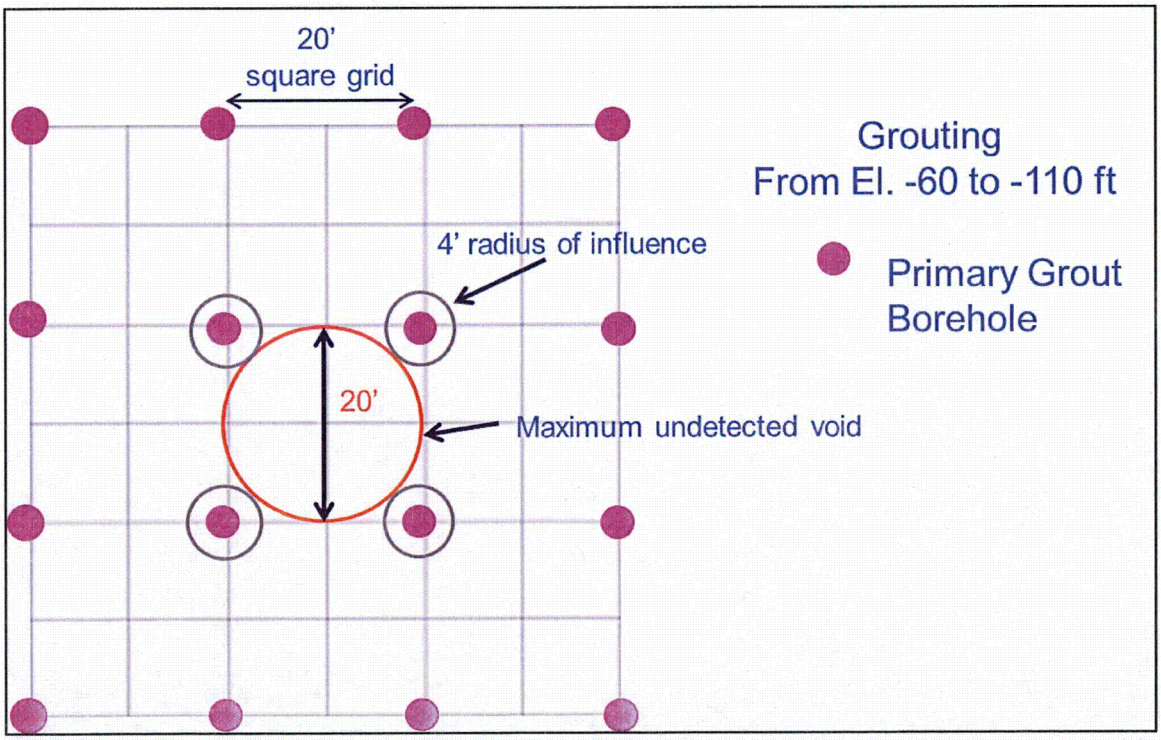
  The maximum length of interpreted tool drop (due to voids and/or voids filled with soft sediments) is limited to 1.5 feet within the Unit 6 & 7 building footprints, and the frequency of encountering an interpreted tool drop is less than 0.5 percent site-wide. These statistics are based on the drilling

conducted during both, the initial and supplemental site investigations (MACTEC, Reference 6; RIZZO, Reference 3)…"

The following methodology is employed to respond to RAI 02.05.04-26:

Instead of developing the critical void size via microgravity methods, which can only detect large voids at depth, the subsurface grouting program, which was planned only for dewatering purposes, will be considered in the determination of constrained void sizes. Part 4 of this response provides an Inspection, Tests, Analyses, and Acceptance Criteria (ITAAC) to ensure that the zone between El. –35 feet and El. –60 feet within the diaphragm walls (the Grouted Zone) will be grouted according to the grout closure criteria that will be developed as part of the grout test program. This grouting will result in any remaining voids in the Grouted Zone having an insignificant impact on the stability of Category I structure foundations (or are structurally insignificant). The void size defined as structurally insignificant will be determined in the grout test program. In addition, for the zone between El. –60 feet and –110 feet within the diaphragm walls (the Extended Grouted Zone), grouting will be performed in every primary grout borehole. Primary grout boreholes will be spaced less than or equal to 20 feet on center. This configuration, as shown in Figure 1, constrains the maximum undetected void size to approximately 20 feet.

Figure 1 Primary Grout Borehole Layout



The void size (20 feet) constrained by the grouting program is conservatively much larger than the estimated void sizes present on site, and has been evaluated in static (settlement and bearing capacity) and dynamic (pseudo-dynamic) analyses. ITAAC are

also provided that when successfully executed will result in any remaining voids in the Grouted Zone being structurally insignificant, and any remaining voids in the Extended Grouted Zone having a maximum equivalent spherical diameter of equal to or less than 20 feet.

Grouting Discussion

Non-safety related grouting will be performed at the site for construction-related groundwater control. A grout plug is proposed between the bottom of the excavation for the Nuclear Island (approximately elevation [El.] –35 feet North American Vertical Datum of 1988 [NAVD 88]) and the bottom of the diaphragm wall (approximately El. –60 feet NAVD 88). The elevation range from El. –35 feet to El. –60 feet is called the "Grouted Zone."

The purpose of the proposed grout plug is to provide a zone of low residual (post-grouting) hydraulic conductivity at the base of the excavation. The Grouted Zone will serve to limit inflows into the excavation and reduce the level of effort required to dewater the excavation during construction to an acceptable/manageable level using dewatering equipment inside the excavation. Due to the high groundwater table and the high hydraulic conductivity of the Key Largo Limestone and Fort Thompson formations at the Turkey Point Units 6 & 7 Site, excess seepage is expected through the rock mass if grouting is not performed.

Various geologic features can provide potential pathways for water flow such as: zones of secondary porosity, fractures, bedding planes, and voids. As discussed in FSAR Appendix 2.5AA, zones of secondary porosity contain vugs on the order of centimeter scale. As discussed in the Response to RAI 2.5.4-25, fracturing and jointing at the site is widely spaced except under the vegetated depressions and drainages, where the Miami Limestone, Key Largo Limestone, and Fort Thompson formations are slightly to moderately fractured as observed within the inclined borings of the supplemental investigation.

Within the slightly to moderately fractured zones, openness of discontinuities varies from tight (no visible separation) to moderately wide (0.03 feet to 0.1 feet), averaging slightly open (less than 0.003 feet). As discussed in the Response to RAI 2.5.4-1, the largest potential voids or sediment infills that were found are limited in size and extent. Based on the field data described above, a void size is defined as equal to or greater than 0.5 feet.

The Response to RAI 2.5.4-26 provides an ITAAC that when successfully executed will result in any remaining voids in the Grouted Zone being structurally insignificant and any remaining voids between El. –60 feet and El. –110 feet having a maximum equivalent spherical diameter of equal to or less than 20 feet, which is accomplished by the grout program. The elevation range from El. –60 feet to El. –110 feet is called the "Extended Grouted Zone."

As discussed in FSAR Subsection 2.5.4.6.2, grouting will be performed within the excavation area for the Nuclear Island. The anticipated grouting layout is shown in FSAR Figure 2CC-239. In general, for the Grouted Zone, grouting will be performed in a

series of split spaced borings starting with primary order grout boreholes, and continuing through secondary order grout boreholes at a minimum. The term "grout borehole" refers to holes drilled for grouting operations and does not necessarily mean that physical samples will be obtained and geologically described. The term "verification boring" refers to holes drilled and water pressure tested where physical samples are obtained to physically and visually assess the suitability of grouting parameters as well as geologically describing the cores. Verification borings not meeting the residual hydraulic conductivity criteria will be pressure grouted. Verification borings meeting the residual hydraulic conductivity criteria will be backfilled with grout.

Grouting will be performed to facilitate construction dewatering and is not classified as safety-related. Mix design, material control, laboratory testing, grout placement, and field testing will be performed under a quality program.

Grouted Zone

For primary grout boreholes in the Grouted Zone, individual grout stages will be grouted to grout stage closure criteria. The grout stage closure criteria will be developed based on results of the grout test program.

Upon completion of the grout stages in the primary grout boreholes, secondary grout boreholes will be drilled and grouted to the grout stage closure criteria. The water pressure test results from verification borings and grout takes from the primary and secondary grout boreholes will be evaluated to determine the need for tertiary and higher order grout boreholes.

For example, if the grout take in an area decreased significantly from the primary to secondary order grout boreholes, tertiary grout boreholes may not be necessary. However, if grout takes in stages in secondary grout boreholes are significant, tertiary grout boreholes will be assigned to further treat the area. In general, higher order grout boreholes will be assigned until the grout takes in the highest order grout boreholes drilled and grouted are acceptably low. When the grout takes have been reduced, the residual hydraulic conductivity of the grout mass will be determined via water pressure tests performed in cored verification borings in the area.

An area of the Grouted Zone will be accepted as complete when the results of verification borings indicate that the residual hydraulic conductivity of the rock mass is equal to or below the target residual hydraulic conductivity. The target residual hydraulic conductivity of the Grouted Zone will be developed based on results of the grout test program.

Extended Grouted Zone

Primary grout boreholes will be extended to El. –110 feet (i.e., just above the interface between the Fort Thompson and Upper Tamiami formations) in order to constrain the maximum undetected spherical void size to approximately 20 feet. The water pressure test results from verification borings and grout takes from the primary grout boreholes will be evaluated to determine the need for secondary grout boreholes.

An area of the Extended Grouted Zone will be accepted as complete when the results of verification borings indicate that the residual hydraulic conductivity of the rock mass is equal to or below the target residual hydraulic conductivity. The target residual hydraulic conductivity of the Extended Grouted Zone will be developed based on results of the grout test program. The Extended Grouted Zone will have a different residual hydraulic conductivity requirement than the Grouted Zone, as the purpose of grouting the Extended Grouted Zone is to constrain maximum undetected void size, not to reduce the hydraulic conductivity of the rock mass.

Grout Test Program

The purpose of the grout test program will be to validate the grout design and grouting techniques, to determine the approximate grout takes for the Key Largo Limestone and Fort Thompson formations, and to determine the grout closure criteria for individual grout stages and for completed areas of the Grouted and Extended Grouted Zones.

The location of the grout test program will not be within the footprint of a safety related structure. The grout test program will be performed prior to production grouting and the Nuclear Island excavation.

The layout for the grout test program will be selected to resemble the planned construction grouting configuration. Grout borehole spacing will be set with regard to the spacing of the dominant geologic features and the construction grouting configuration. Grout borehole orientations and inclinations will be selected to promote intersections with the dominant fractures and bedding features in the area of the work. Since the fractures in the Key Largo Limestone and Fort Thompson formations range from vertical and subvertical to around 40 degree dip, it is anticipated that the inclination of the grout boreholes will be adjusted to best intercept the dominant features in the treatment area.

It is anticipated that on the order of ten primary grout boreholes will be drilled for the grout test program, with a spacing of approximately 20 feet. It is anticipated that on the order of five secondary grout boreholes will be drilled for the grout test program, and will be offset from primary grout boreholes such that a secondary grout borehole is at the center of the square formed by four adjacent primary grout boreholes. Verification borings will be drilled at various locations within the grouted area to measure residual hydraulic conductivity of the rock mass and to physically and visually assess the suitability of grouting parameters. It is anticipated that five verification borings will be drilled for the grout test program.

The grout test program will be used to optimize and finalize the grouting and dewatering specifications, including:

- Spacing for primary and secondary grout boreholes. The ITAAC provided in the Response to RAI 2.5.4-26 states that primary grout boreholes will be spaced less than or equal to 20 feet on center. Spacing of primary and secondary grout boreholes may be reduced based on results from the grout test program.
- Suitability of the formation for grouting via downstages, upstages, or a combination of upstages and downstages. Upstage grouting is a method whereby packers or expansion plugs block off preselected portions of the grout boreholes in ascending stages while those portions are being grouted. In this method, grout boreholes are drilled to their full depth, pressure tested, and grouted in successive stages from the bottom up. Alternatively, downstage grouting is the grouting of progressively deeper zones in stages, with the deeper zones accessed by drilling through previously injected grout. Effectiveness of the staging method will be determined by grout borehole conditions during drilling including grout borehole instability during or after drilling, loss of circulation of drill fluid, difficulties setting packers in grout boreholes during grouting, and by the level of improvement of grout borehole conditions after grouting has been performed in an area.
- Inclination of grout boreholes (vertical or inclined). As described above, grout borehole orientations and inclinations will be selected to promote intersections with the dominant fractures and bedding features in the area of the work. It is anticipated that the inclination of the grout boreholes will be adjusted during the grout test program to best intercept the dominant features.
- Effective grout mixes using locally available materials and water sources. Multiple grout mixes will be identified for high and low mobility grout. Grout mixes will be tested to assess their mobility, stability, and durability. Different grout mixes will be considered for the Grouted Zone and the Extended Grouted Zone. As discussed in the Response to RAI 2.5.4-31, the sulfate values measured from 24 water samples range from 2280 ppm to 4400 ppm, resulting in a median value of about 3800 ppm, or close to 0.4 percent by weight. This classifies the concrete exposure to sulfate attack as Class 2 exposure according to ACI 201.2R-08 (Reference 3). The amount of grout pumped into potential voids is expected to be minimal (no physical evidence indicating large voids), and variable across the site. Stability analyses do not consider any increase in strength or stiffness of the rock mass due to the presence of the grout.
- Drilling and flushing of grout boreholes. The grout boreholes will be advanced using water flushed, rotary, or rotary percussive drilling. The primary grout boreholes in an area will be drilled and grouted before the drilling proceeds on the secondary grout boreholes. Grout borehole rock drilling will be monitored for penetration rate, down thrust pressure, rod torque, drilling fluid pressure and flow. Rock mass data will be collected using a drilling parameter recorder.
- Injection rates and pressure. Grouting pressures will be selected using the grouting intensity method pressure-volume curves from Lombardi (Reference 4) in combination with more traditional industry practice (e.g., the US Army Corps of Engineers, Reference 5, uses 0.5 psi per foot of overburden and 1.0 psi per foot of

rock). Injection rates will be dictated by the ability to reach and maintain the target pressure. Injection volume limits will be optimized in the test program by use of the pressure-volume curves and evaluating the grout travel distance. Additionally, target pressures will be evaluated from the perspective of a potential hydraulic fracture in the grouted rock and the peak allowable grouting pressure will be established for the site.

- Grouting conditions will be evaluated by using computer controlled real time monitoring of grout injected volumes, injection pressures, injection flow rates, grout mix changes and automatic recording of data.
- For primary grout boreholes in the Grouted Zone, initially individual grout stages will be grouted to "absolute refusal" grout stage closure criteria. Absolute refusal for the stage will be defined as zero (no measureable) flow of grout into the rock formation with the injection pressure maintained at the target pressure for the target duration of time. After enough (not more than eight adjacent) of the primary grout boreholes have been grouted using these criteria, the grout stage closure criteria may be adjusted based on grout takes, the results of water pressure tests from verification borings, evaluation of available boring data, and the target residual permeability. If the grout take for a stage is significant, closure will be achieved by using progressively thicker mixes that have reduced mobility. If necessary to reach closure, grouting may be stopped temporarily to allow grout to set and then resumed.
- Verification borings will be drilled and cored for the performance of water pressure tests to measure the residual hydraulic conductivity of the rock mass and to physically and visually assess the suitability of grout borehole spacing and inclination. Acceptance criteria for a completed area of the Grouted Zone will be based on water pressure testing performed in verification borings. For example, a target area residual hydraulic conductivity of 10 Lugeons or less is generally reasonable. The acceptance criteria will be confirmed to be adequate for the site based on the results from the grout test program. The Extended Grouted Zone will have a different residual hydraulic conductivity requirement, as the purpose of grouting this zone is to constrain maximum undetected void size.

Part 1)

In response to Part 1, a sensitivity analysis is performed, including static (settlement and bearing capacity) and pseudo-dynamic evaluations, to demonstrate that the void size constrained by the grouting program is not critical to the stability of subsurface materials and the integrity of SSCs.

Void Sizes and Depths Considered

The analysis considers the Grouted Zone to be completely grouted and void free. The Extended Grouted Zone will be grouted partially at primary grout holes spaced 20 feet apart. This spacing, with a conservative grout radius of influence of 4 feet, corresponds to a sphere shaped void with a maximum 20 feet equivalent diameter (as shown in Figure 1).

A very extreme case of a tunnel-shaped (i.e., cylindrical) void with a 20-foot diameter circular cross-section is considered where the void extends east-west across the nuclear island with the top of the void just below the grout plug (El. –60 feet). This direction is selected since the maximum dynamic bearing demand occurs under the west edge of the shield building and is primarily due to the response to the east-west component of the earthquake (DCD Tier 2, Subsection 2.5.4.2).

The sensitivity analysis considers extremely unlikely and conservative cases that are only assessed to show the safety margin provided by the rock mass; these cases are highly unlikely and are not for design purposes.

Modeling Approach

For the analysis, the 3D finite element model (as presented in the Response to RAI 02.05.04-19) is updated for the void case presented above. Best estimate material properties (FD1 properties for rock layers) are used for this analysis, as described in the Response to RAI 02.05.04-19, and the software PLAXIS 3D foundation (referred to as PLAXIS 3D hereafter) is used for the modeling.

The postulated void is assumed to be water-filled, and is therefore modeled with the same pore pressures as the surrounding rock.

The model considers a construction sequence that includes the following activities:

- Initial gravity loading (without the void),
- Gravity loading (with the void),
- Dewatering,
- Excavation and fill placement,
- Loading, and
- Rewatering.

The void is not considered in the initial gravity loading phase because it would have developed over time; further, inserting the void in the second phase allows for an evaluation of any points reaching Mohr-Coulomb failure due to the presence of the void independent from the other construction activities.

Nuclear Regulatory Commission (NRC) RG 1.132 Appendix D states that, "*Where soils are very thick, the maximum required depth for engineering purposes, denoted d_{max}, may be taken as the depth at which the change in the vertical stress during or after*

construction for the combined foundation loadings is less than 10% of the effective in situ overburden stress." The analysis depth of El. –450 feet, which is greater than 2B (B = the least dimension of the foundation), was assumed to be adequate to meet the aforementioned criterion. In situ initial overburden effective vertical stress at the bottom of the model is compared to the effective vertical stress at the bottom of the model for each phase. The changes in effective vertical stresses are less than 10 percent of the effective in situ stress for each phase, demonstrating that the model depth is appropriate.

As discussed in the Response to RAI 02.05.04-19, the plan dimensions considered in the model are 1724 feet by 1396 feet. The lateral boundary conditions were checked by confirming that the horizontal stresses at the edge of the model are in agreement with horizontal stresses calculated by hand. Effective horizontal stresses are calculated using Equations 1 and 2, below (Reference 6).

$$\sigma'_h = K_0 \times \sigma'_v \qquad \text{Equation 1}$$

Where,

σ'_h = the effective horizontal stress,

σ'_v = the effective vertical stress,

and K_0 = the at rest earth pressure coefficient

$$K_0 = \frac{v}{1-v} \qquad \text{Equation 2}$$

Where,

v = Poisson's ratio

Figures 2 and 3 show a comparison of the effective horizontal stresses at the east (effective horizontal stress = σ'_{xx} since the plain strain conditions prevail in xx-direction) and north (effective horizontal stress = σ'_{zz} since the plain strain conditions prevail in zz-direction) boundaries of the model with the horizontal stresses calculated by hand. Model stresses are shown for the initial phase and loading phases. The average differences per layer between the effective horizontal stresses in the initial and loading phases are provided in Table 1. Figures 2 and 3 generally demonstrate good agreement between the effective horizontal stresses at the model boundary and the horizontal stresses calculated by hand, confirming that the model extent is appropriate.

The lateral boundary conditions were also checked in the Response to RAI 02.05.04-19 by demonstrating that the total displacement at the corner of the model is less than 0.1 inches. Additionally, a sensitivity analysis was performed for the best estimate model (presented in the Response to RAI 02.05.04-19), where the model boundaries were extended such that the distance from the buildings to the edge of the model is approximately twice that of the best estimate model. Maximum settlement predicted for the buildings does not vary with the extended boundaries, also confirming that the lateral extent of the best estimate model is adequate.

Figure 2 Effective Horizontal Stresses – East Model Boundary

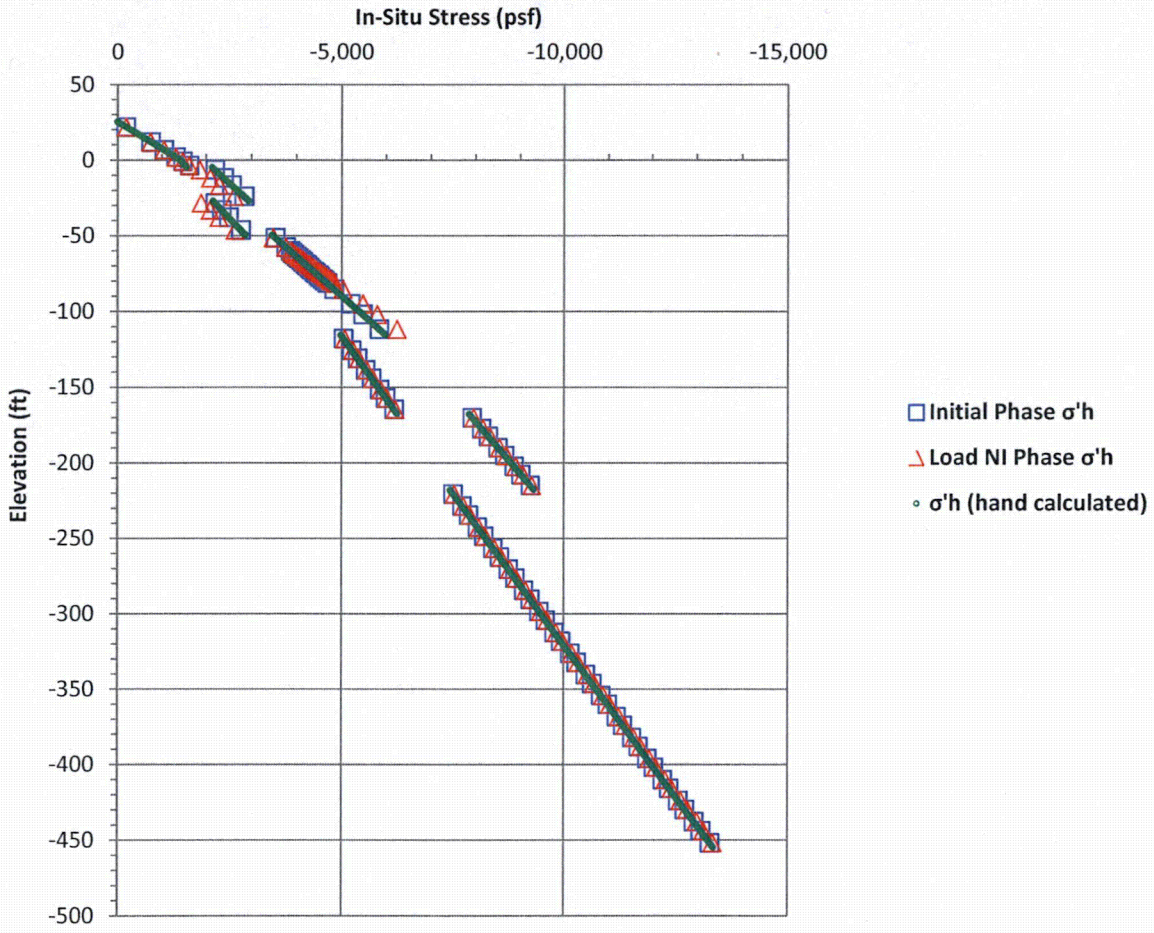


Figure 3 Effective Horizontal Stresses – North Model Boundary

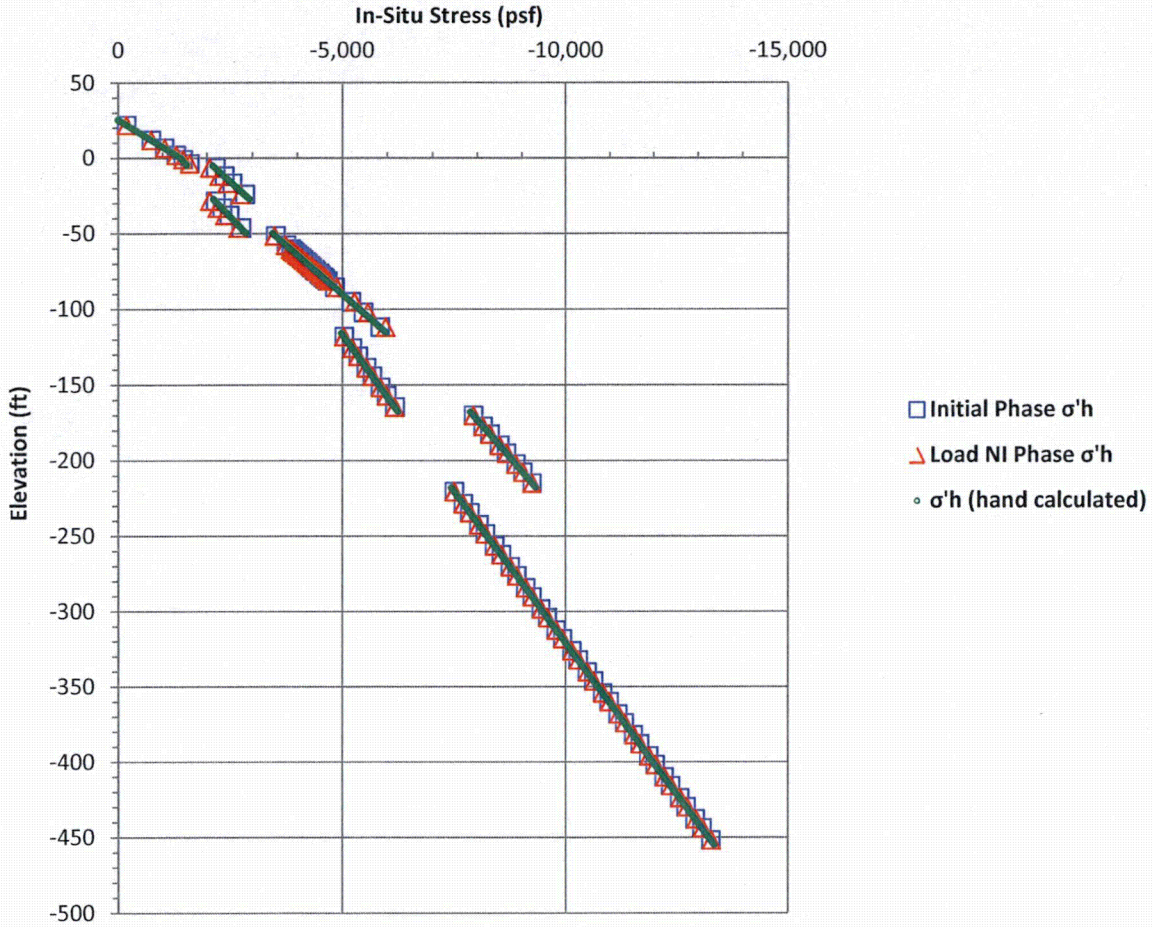


Table 1
Percent Difference between Effective Horizontal Stresses in the Initial and Loading Phases

Formation	North Model Boundary	East Model Boundary
Backfill	1%	1%
Miami Limestone	5%	12%
Key Largo Limestone	4%	10%
Fort Thompson Formation	1%	2%
Upper Tamiami	0%	0%
Lower Tamiami	0%	0%
Peace River	0%	0%

Figures 4 through 6 show the PLAXIS3D model. The 3D mesh is refined to the extent possible in the area surrounding the void. The total number of elements is 103,136.

Figure 4 PLAXIS3D Model

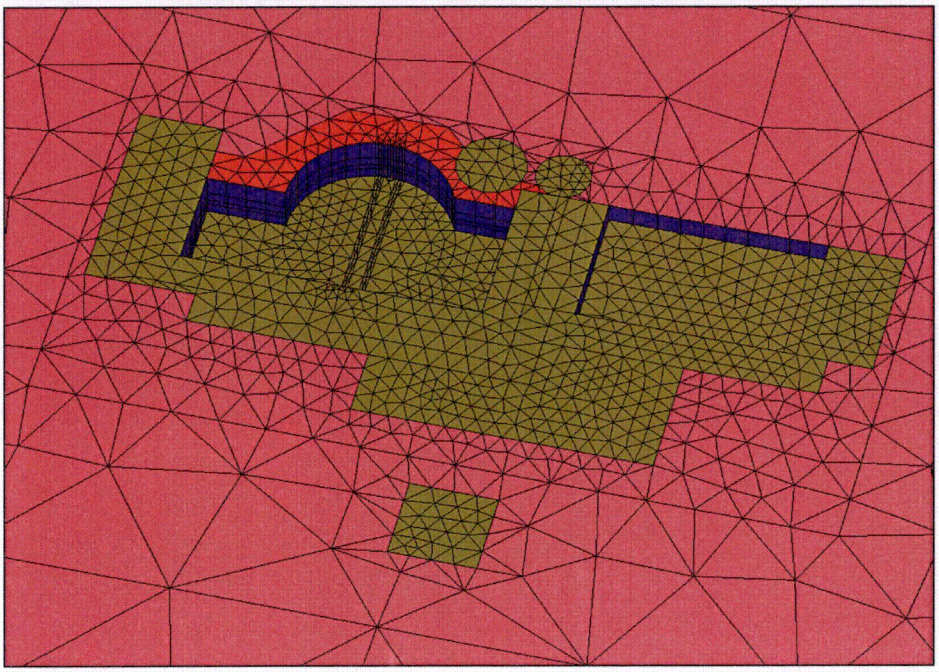


Figure 5 PLAXIS3D Model – Plan View

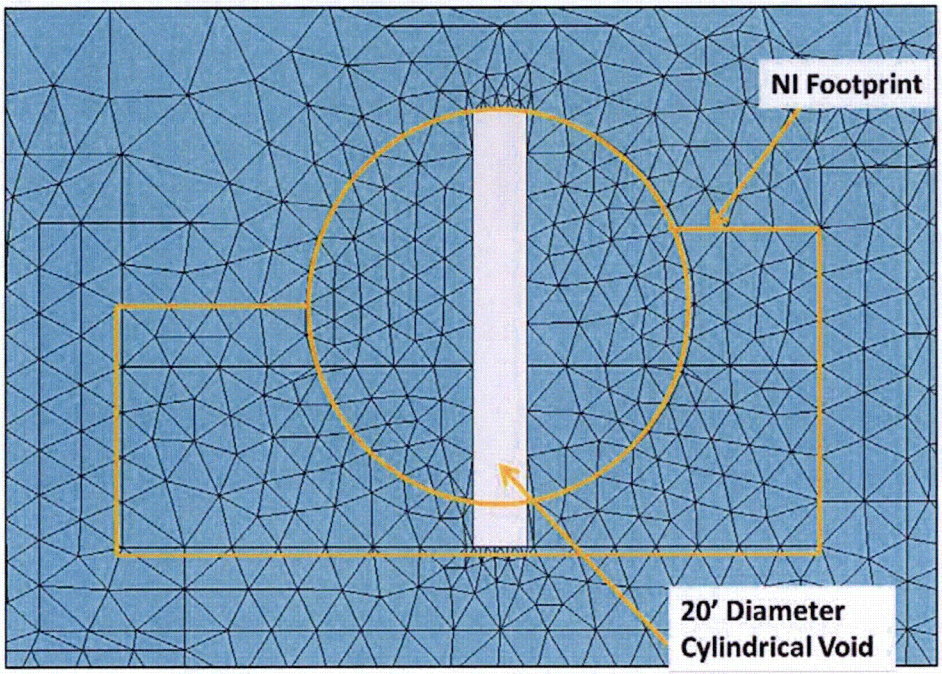
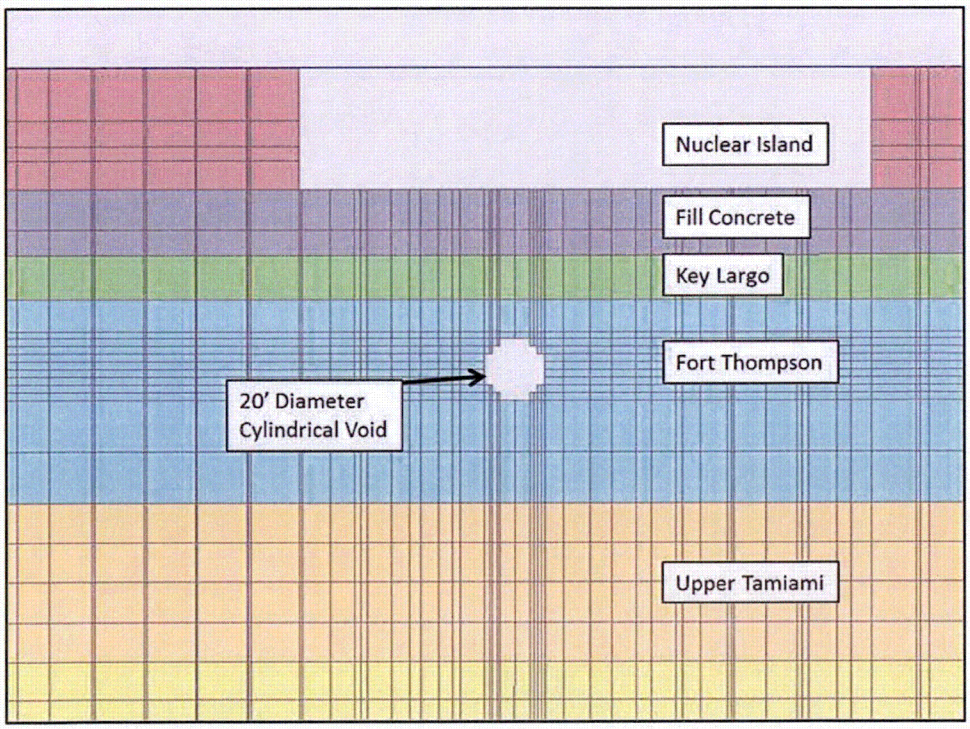


Figure 6 PLAXIS3D Model – Cross-Section



Settlement

Vertical deformation due to loading is evaluated in the PLAXIS3D finite element models to determine the impact of the potential void on the settlement of the nuclear island. Differential settlements are calculated and the results are compared to the DCD requirements.

Bearing Capacity

To determine the impact of the potential void on the bearing capacity, the model is incrementally loaded up to much higher loads than the actual building loads and a load displacement curve is developed. This curve is used to evaluate the bearing capacity.

Concrete Fill Properties

In order to assess potential tension in the concrete, the concrete is assumed to be a Mohr-Coulomb material in the PLAXIS3D model with a friction angle of 0 degrees, cohesion of 108,000 psf, and tensile strength of 21,600 psf based on Equation 3 (Reference 6), Equation 4 (Reference 7), and a compressive strength of 1500 psi. The tensile strength calculated based on Equation 4 is conservative compared to the tensile strength calculated according to ACI 207.1R-05 (Reference 10).

$$Cohesion = \frac{Compressive\ Strength}{2} \qquad \text{Equation 3}$$

$$Tensile\ Strength \approx 0.1 \times Compressive\ Strength \qquad \text{Equation 4}$$

Pseudo-Dynamic

To consider the impact of the potential voids on under dynamic conditions, dynamic bearing pressures from the SASSI model are converted to equivalent (approximately) static loads and applied to the PLAXIS 3D model.

The forces from the dynamic bearing pressures are summed up and distributed uniformly over areas of the eastern (maximum uplift) and western (maximum compression) halves of the nuclear island. The maximum uplift bearing pressures as obtained from the upper bound, lower bound, and best estimate cases are applied on the eastern half of the nuclear island, whereas the maximum compressive bearing pressures as obtained from the upper bound, lower bound, and best estimate cases are applied on the western half of the nuclear island, such that the maximum overturning moment is applied on the western edge of the nuclear island.

This approach is very conservative because

- Nodal maximum bearing pressures are used regardless of their time step (note that maximum pressures for each node correspond to different time steps, i.e., they do not happen at the same time).
- Maximum compressive pressures and tensile pressures are applied at the same time to maximize the overturning moment.

Additionally, a case is considered where the load combinations are multiplied by a safety factor of 2. Table 2, below shows the total loads considered (static and pseudo-dynamic). The sum of the static load and the seismic uplift pressure is negative, if the overall pressure is compressive.

Table 2
Total Static and Pseudo-Dynamic Loads (ksf)

Building	Multiplier of 1		Multiplier of 2	
	West Half of NI	East Half of NI	West Half of NI	East Half of NI
Shield Building	-16.2	-8.1	-32.3	-4.3
North Auxiliary Building	-9.5	-1.4	-18.9	2.4
South Auxiliary Building	-12.1	-4.0	-24.1	-0.2

Note: Negative sign indicates compressive loads, positive sign indicates uplift loads.

The bearing pressures corresponding to the combination of dead loads and the maximum moment are checked against the bearing capacity of the concrete fill. The ultimate bearing capacity for the concrete fill is estimated to be 1275 psi (184 ksf) using Equation 5 (Reference 8) and a compressive strength of 1500 psi.

$$Ultimate\ Bearing\ Capacity = 0.85 \times f'c \qquad \text{Equation 5}$$

Where,

f'c = compressive strength

Results

All model results presented are for the case with a tunnel-shaped (i.e., cylindrical) void with a 20-foot diameter circular cross-section. The tunnel-shaped (i.e., cylindrical) void is considered to be more critical than a smaller 20-foot diameter spherical void, or a distribution of spherical voids.

Soil layers are modeled using Mohr-Coulomb material properties. The Mohr's failure stress circle is controlled by the principal stresses and strength properties (friction angle and cohesion) of the material, as shown by the governing equations below (Reference 6).

$$s = c + \sigma \tan(\varphi) \qquad \text{Equation 6}$$

$$\sigma_1 = \sigma_3 tan^2\left(45° + \frac{\varphi}{2}\right) + 2c\tan\left(45° + \frac{\varphi}{2}\right) \qquad \text{Equation 7}$$

$$\sigma_3 = \sigma_1 tan^2\left(45° - \frac{\varphi}{2}\right) - 2ctan(45° - \frac{\varphi}{2}) \qquad \text{Equation 8}$$

Where,

s = shear strength

c = cohesion

φ = friction angle

σ_1 = major principal stress

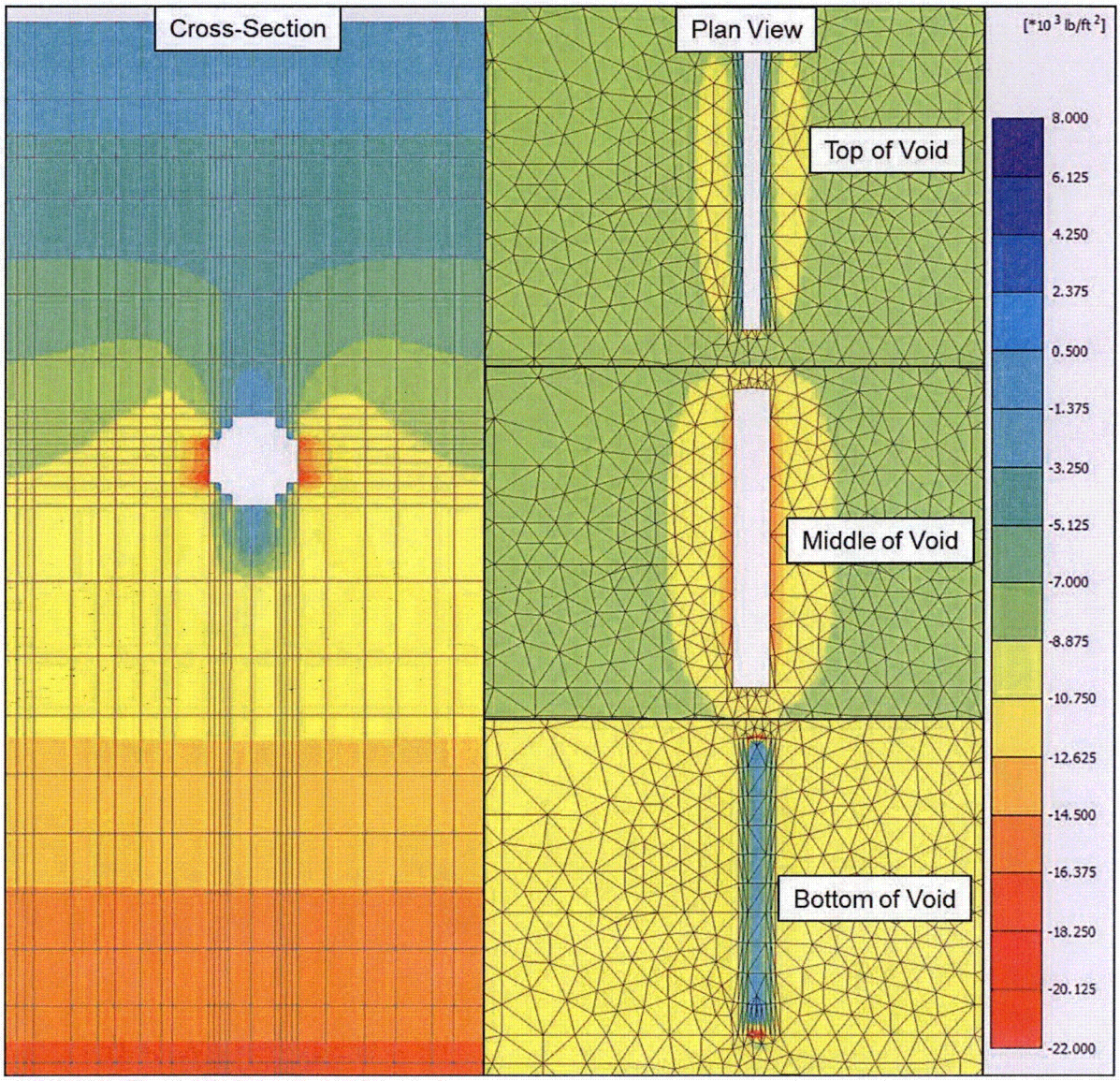
σ_3 = minor principal stress

Results are evaluated in terms of effective vertical stresses, shear stresses, and plastic points (points reaching the Mohr-Coulomb failure envelope). Effective vertical stress and shear stress plots are utilized to identify the high-stress areas (hot spots). In other words, they are used to develop a "feel" for the behavior. The static stability check is actually governed by the following four factors:

1. Accumulation of plastic points indicating a local (e.g., around the void) or global (e.g., bearing capacity) failure.
2. Deformations exceeding DCD limits.
3. Concrete layer experiencing tension failure.
4. Bearing capacity with FOS<3.

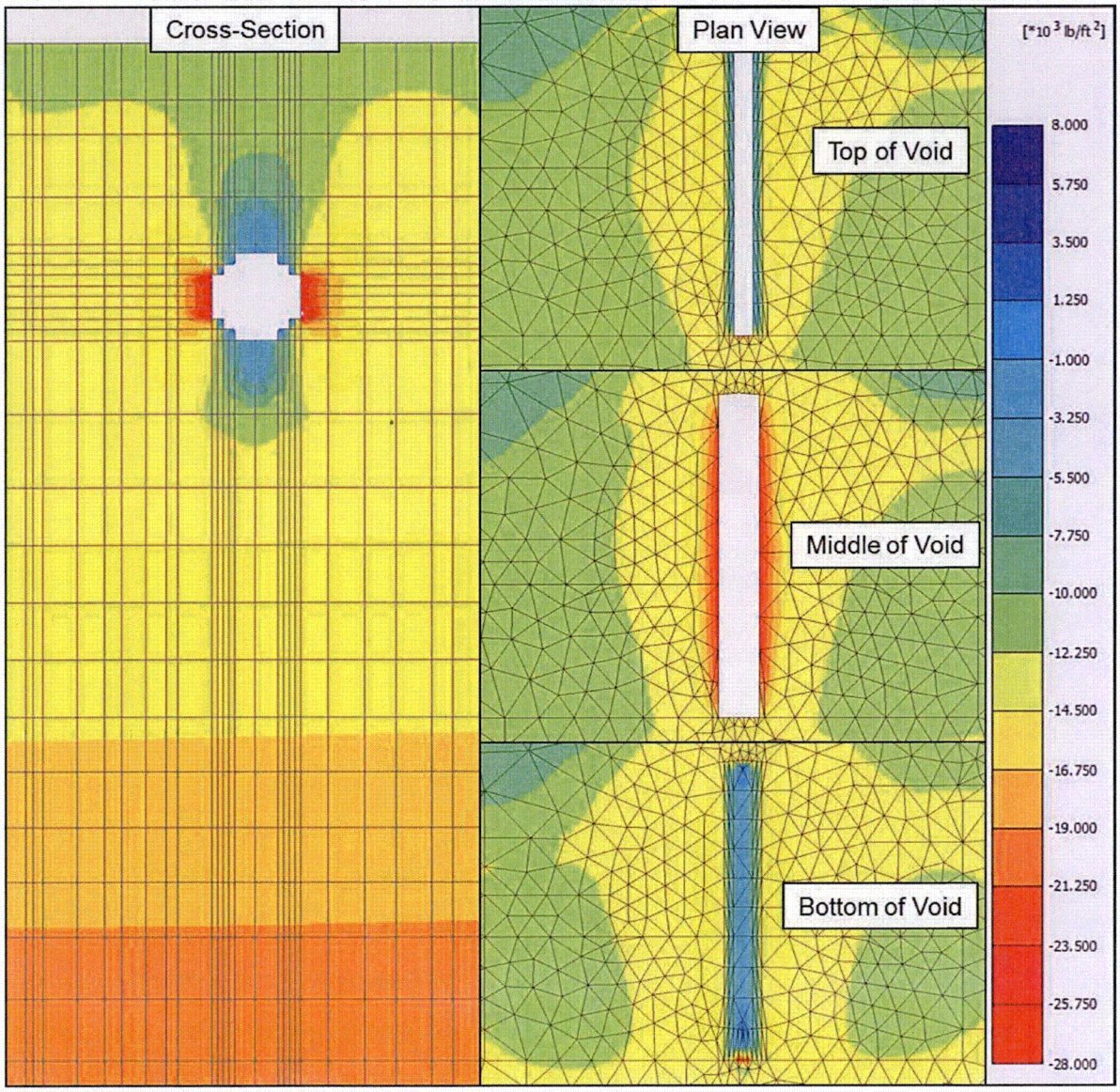
Effective vertical stresses from the PLAXIS3D model are shown in Figures 7 and 8, below. Negative effective stresses presented in Figures 7 and 8 represent compression, while positive effective stresses represent tension. Larger compressive stresses are observed on the sides of the void indicating the expected deformation of the void under building loads. Effective vertical stresses are highest at the edges of the voids in the models, as anticipated, since the stresses are expected to concentrate around the voids particularly. Some effective stresses are larger in areas of sharp discontinuities, which are artificial products of finite element modeling. Stresses at these locations are not expected to be this high, since the realistic stress distribution is expected to be smoother.

Figure 7 Effective Vertical Stresses from PLAXIS3D (Gravity Loading with the Void)



Note: Negative effective stresses represent compression, while positive effective stresses represent tension.

Figure 8 Effective Vertical Stresses from PLAXIS3D (Static Loading Phase)



Note: Negative effective stresses represent compression, while positive effective stresses represent tension.

Shear stresses from the PLAXIS3D models are shown in Figures 9 and 10, below. Shear stresses are highest at the edges of the voids, as expected.

Figure 9 Shear Stresses from PLAXIS3D (Gravity Loading with the Void)

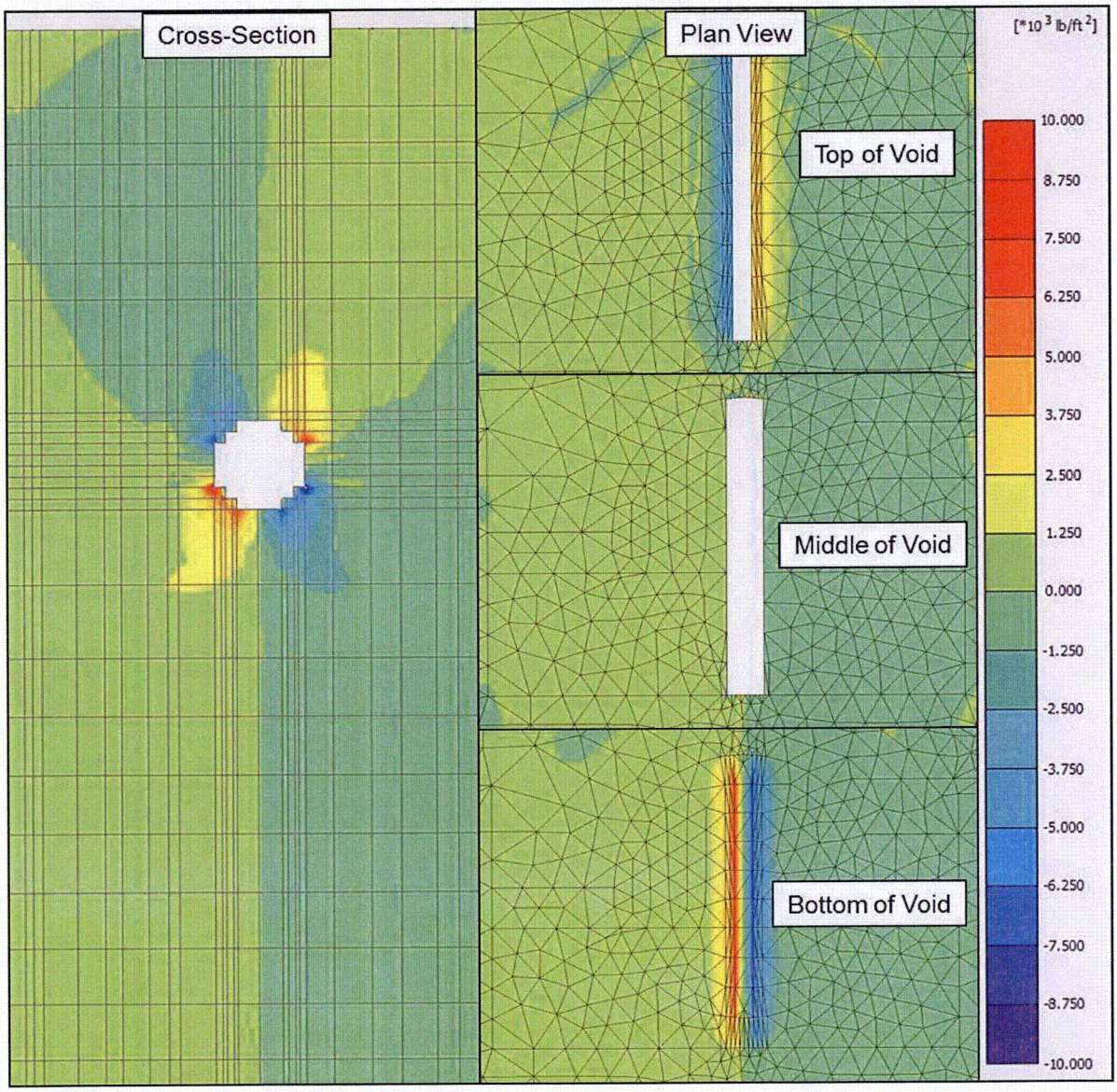
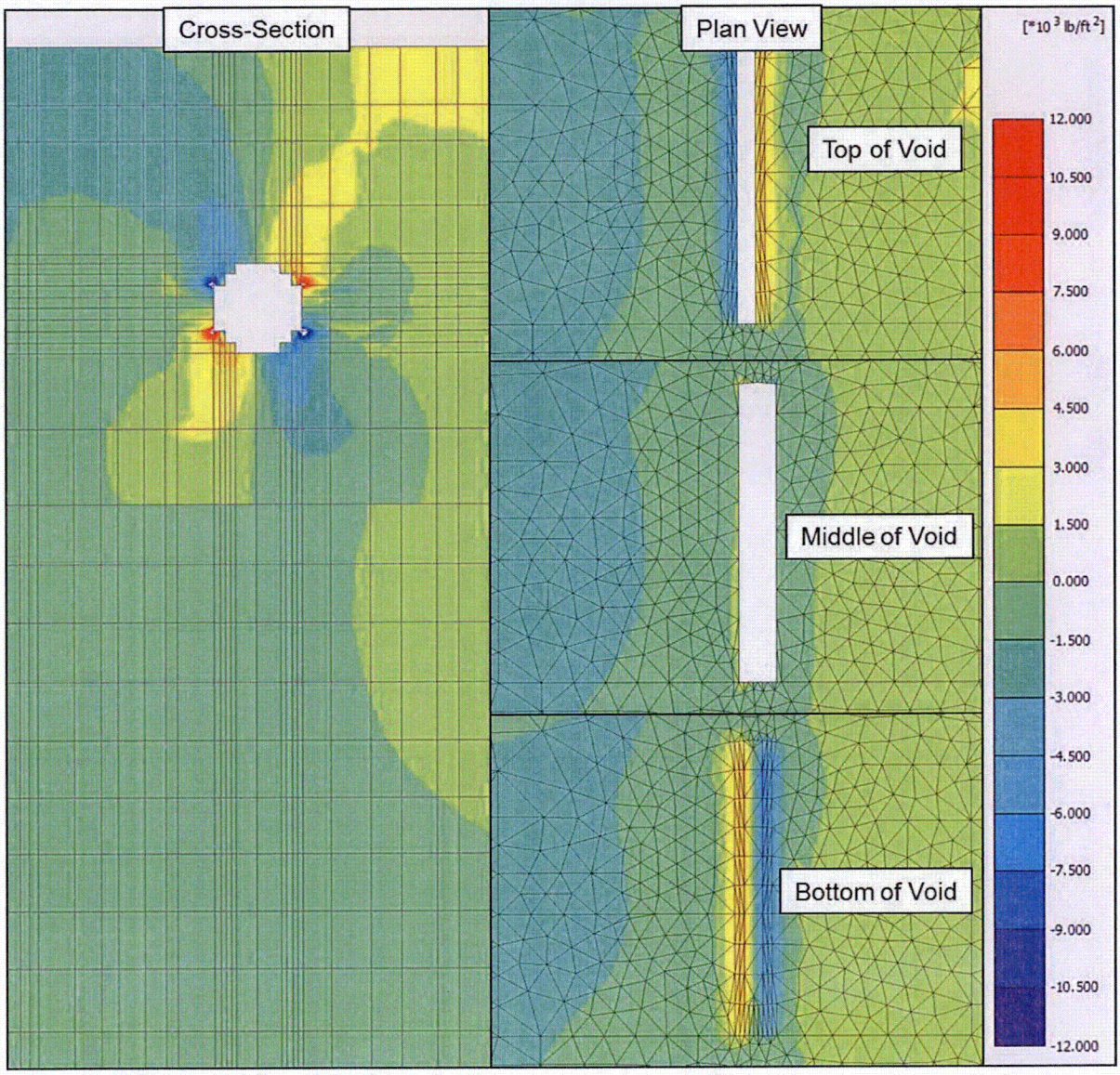
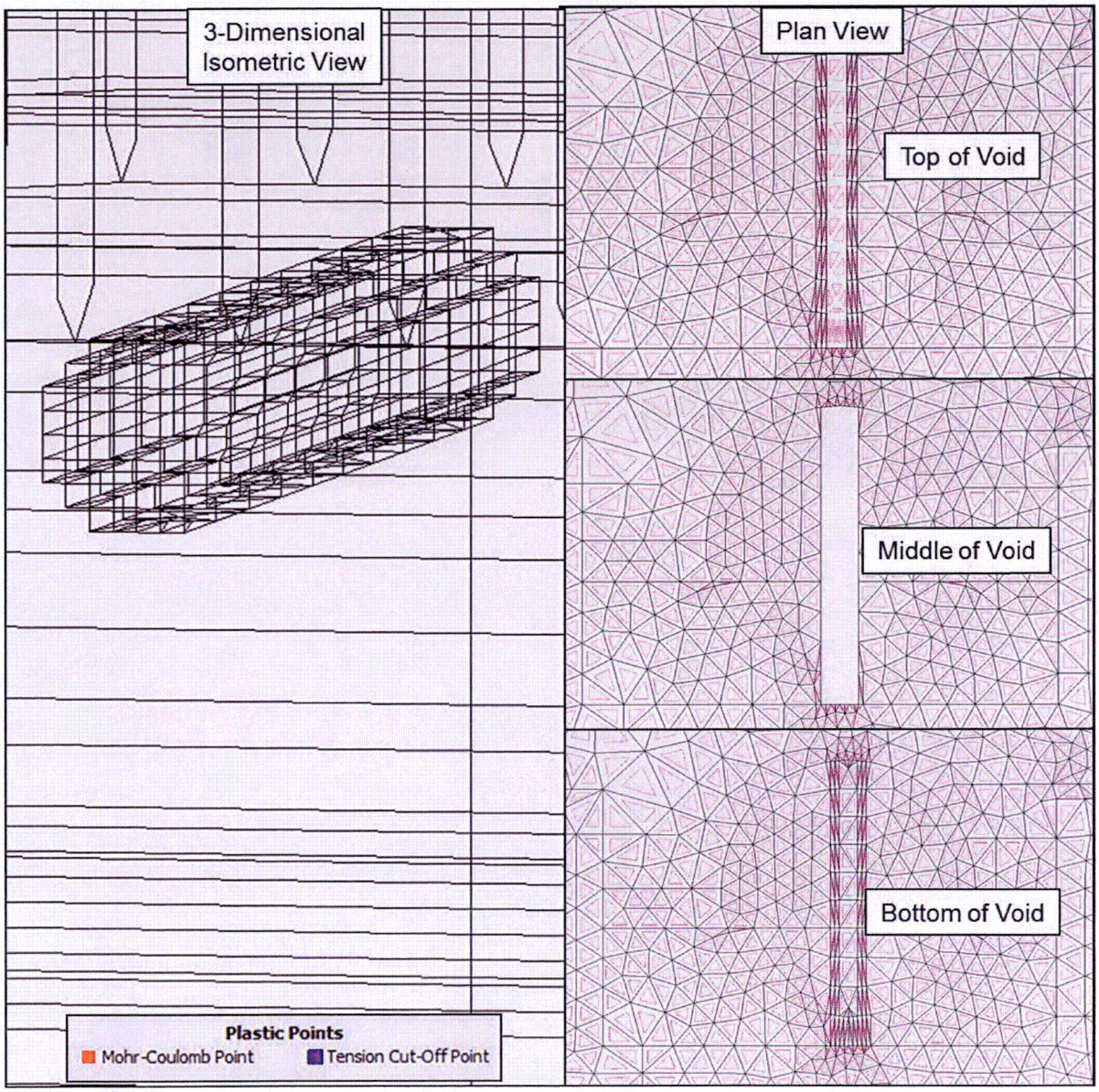


Figure 10 Shear Stresses from PLAXIS3D (Static Loading)



Yield at any point is considered to occur if the stress state reaches the Mohr-Coulomb failure envelope. As shown in Figure 11, there are no plastic points (indicated by red squares) or tension cut-off points (indicated by blue squares) near the void location, indicating that the rock mass surrounding the void is not experiencing compressive failure according to Mohr-Coulomb failure envelope or points where the tensile load exceeds the tensile capacity. No tension cut-off points are identified within the concrete fill, i.e., the tension in the concrete fill is less than the tensile capacity of the concrete fill.

Figure 11 Plastic Points PLAXIS3D (Static Loading)



Another useful parameter to consider is the relative shear stress, which is a measure to define how close the stress state is to the Mohr-Coulomb failure envelope. Relative shear stresses are defined in Equation 9 (Reference 9).

$$\tau_{rel} = \frac{\tau_{mob}}{\tau_{max}} \qquad \text{Equation 9}$$

Where,

τ_{rel} = relative shear stress,

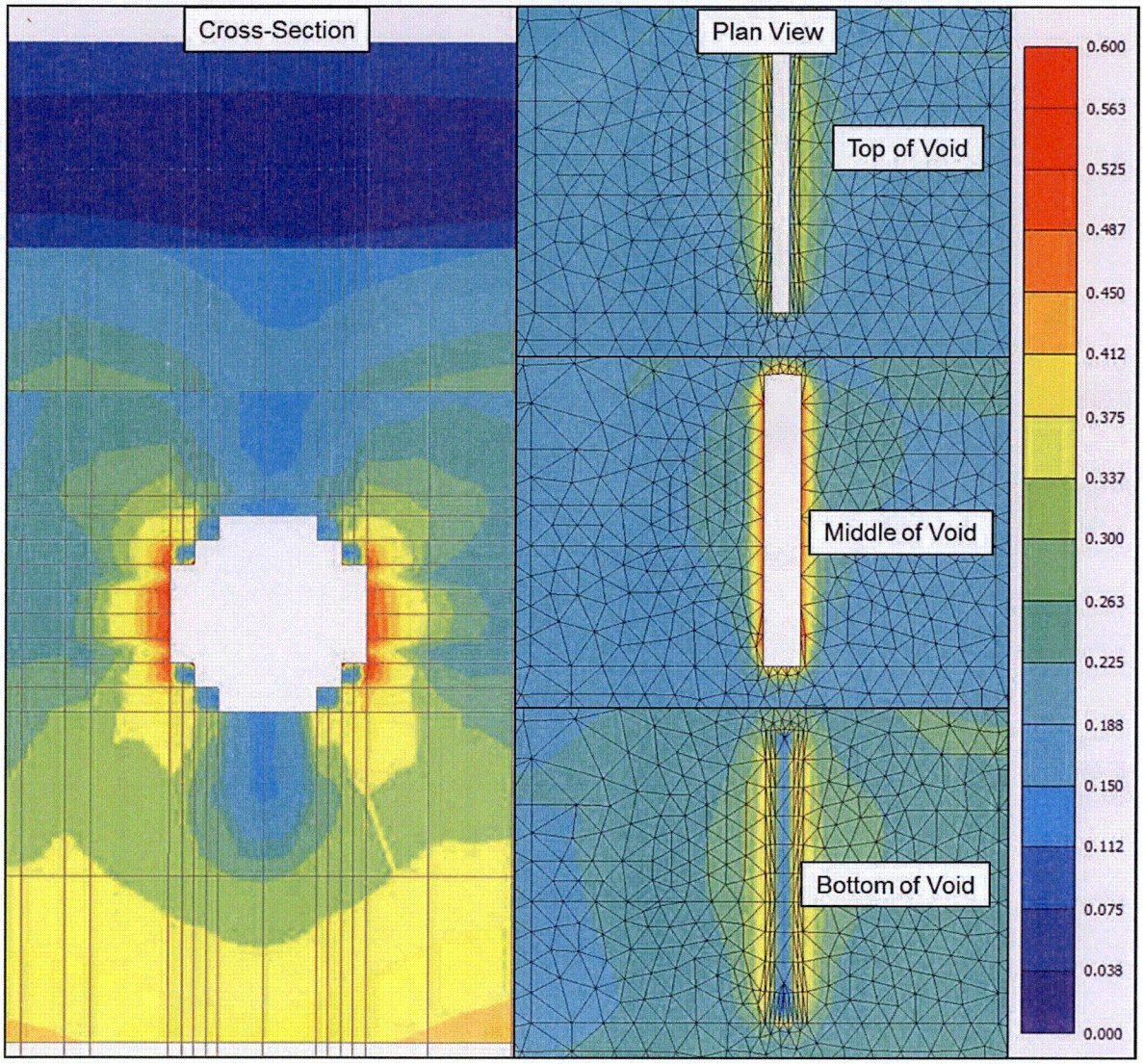
τ_{mob} = mobilized shear strength (maximum value of shear stress),

τ_{max} = maximum value of shear stress for the case where the Mohr's circle is expanded to touch the Coulomb failure envelope while keeping the center of Mohr's circle constant.

Based on Equation 9, relative shear stresses provide a measure of margin compared to Mohr-Coulomb failure. For example, if the relative shear stress is equal to 1, then that location is marked with a plastic point. If the relative shear stress is much less than 1, the point is not close to the Mohr-Coulomb failure envelope. As shown by Figure 12, the rock surrounding the void indicates relative shear stresses much less than 1.

In conclusion, the presence of a 20-foot wide tunnel as modeled here does not present stability concerns, i.e., no subsurface collapse is anticipated.

Figure 12 Relative Shear Stresses (Static Loading)



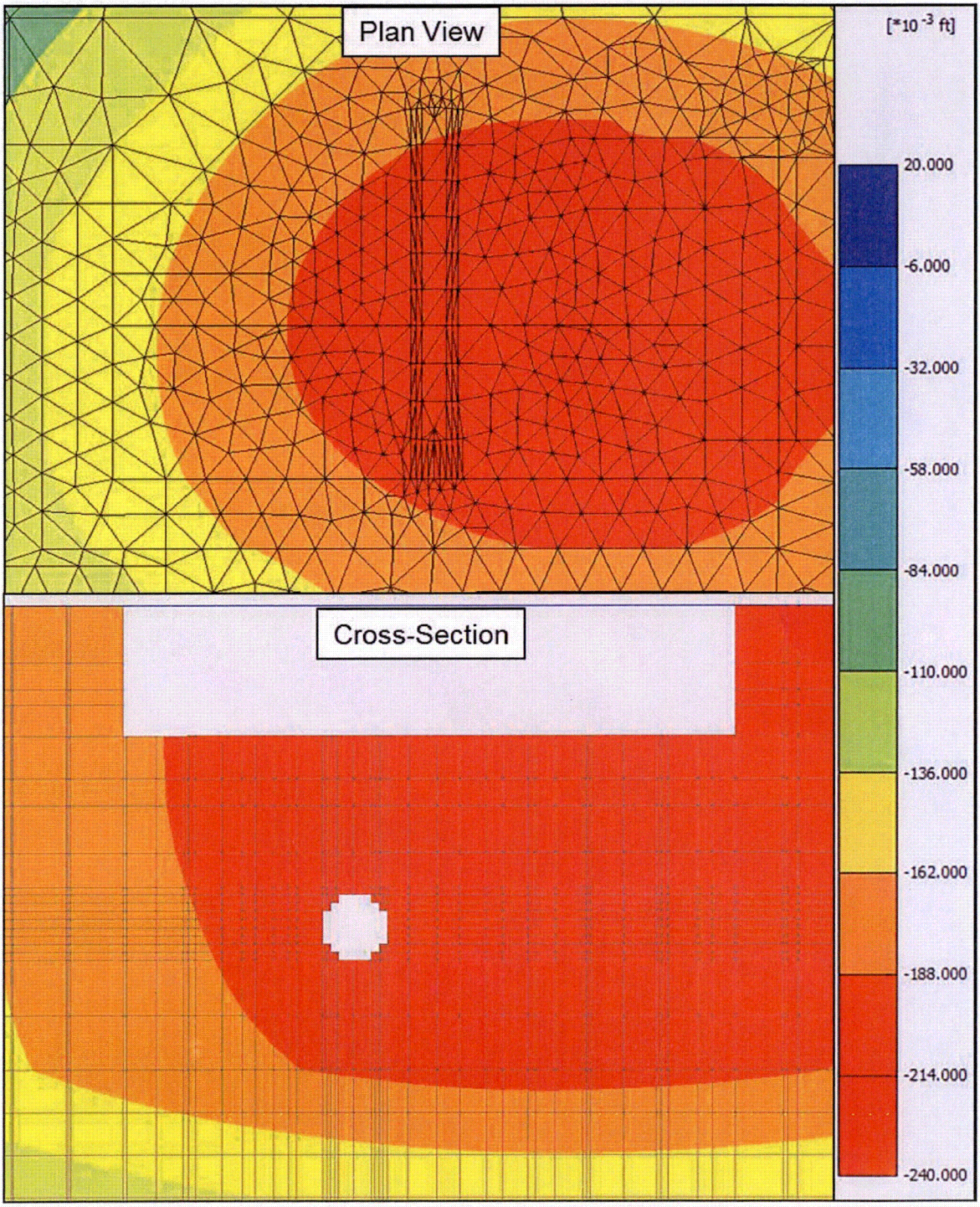
The total and differential settlements from the PLAXIS3D model are presented in Table 3, below. For reference, Table 3 also includes the DCD requirements and the settlements for the model without voids (as presented in the Response to RAI 02.05.04-19). These results show that the presence of the void has no impact on settlement, and that the DCD criteria are still met. The total vertical deformation (loading nuclear island phase) from the PLAXIS3D model is shown in Figure 13, below. Therefore the void cases considered are not critical to the settlement of safety related structures.

Table 3
Total and Differential Settlements Compared to DCD Criteria

	Differential Across Nuclear Island Foundation Mat (3)(4) (inch per 50 feet)	Total for Nuclear Island Foundation Mat (3)(4) (inch)	Differential Between Nuclear Island and Turbine Building(1)(3)(4) (inch)	Differential Between Nuclear Island and Other Buildings(1)(2)(3)(4) (inch)
DCD Requirement	0.5	6	3	3
Plaxis 3D (No Void)	0.20	2.53	0.79	1.60
Plaxis 3D (20' Diameter Tunnel Void)	0.20	2.52	0.79	1.61

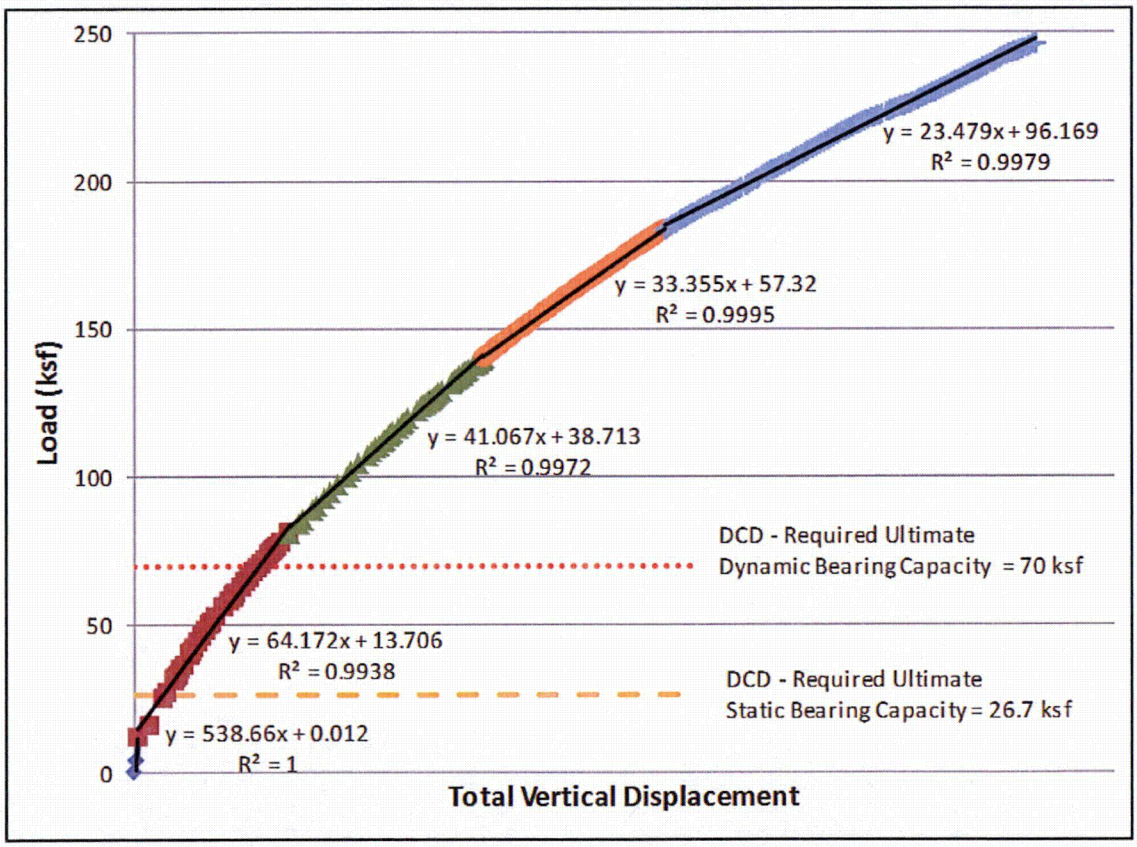
1. Differential settlement is measured at the center of the nuclear island and the center of adjacent structures.
2. Maximum differential settlement occurs between nuclear island and radwaste buildings.
3. Settlements presented exclude the rewatering phase.
4. Settlements are presented to two decimal places to show that differences in settlement between the Plaxis 3D model (No Void) and Plaxis 3D model (20' Diameter Tunnel Void) are negligible. These negligible differences in settlement are considered to be within the variability associated with the modeling (such as mesh differences).

Figure 13 Total Vertical Deformation PLAXIS3D (Static Loading)



To evaluate bearing capacity, the nuclear island load was incrementally increased and a load displacement curve was developed, as shown in Figure 14 which also shows ultimate static (Factor of safety[FOS]=3)and dynamic(FOS=2) bearing capacities required by the DCD. The required ultimate static and dynamic bearing capacities from the DCD are within the linear elastic range of the load displacement curve. Actual bearing capacity as defined by the load-displacement curve shown in Figure 14 is much higher than DCD limits. The load-displacement curve indicates that the significant reduction in stiffness, which can be defined as the ultimate bearing capacity, does not occur for loads up to 250 ksf, i.e., the actual bearing capacity according to Figure 14 is higher than 250 ksf. Please note the bearing capacity definition used here is not a serviceability limit and is rather an ultimate state, thus, the deformation levels are not relevant in this analysis.

Figure 14 Load Displacement Curve



As indicated by the results shown in Figures 11 through 14 and in Table 3:

- No plastic points or tension cut-off points are observed during the loading phase (1x bearing demand),
- The factor of safety for bearing capacity is greater than 3,
- The DCD criteria for settlement are met, and

- The tensile capacity of the concrete fill is not reached.

In summary, the void size constrained by the grouting program has been demonstrated to not be critical to the static stability of safety related structures.

Response under Dynamic Loads

As shown in Figures 15 and 16, under pseudo-dynamic loading (multiplier of 1 and multiplier of 2) there are no plastic points (indicated by red squares) or tension cut-off points (indicated by blue squares) near the void location indicating that the rock mass surrounding the void is not experiencing compressive failure according to Mohr-Coulomb failure envelope or points where the tensile load exceeds the tensile capacity. No tension cut-off points are identified in the concrete fill, i.e., the tension in the concrete fill is less than the tensile capacity of the concrete fill.

Figure 15 Plastic Points PLAXIS3D
(Static and Pseudo-Dynamic Loading, Multiplier of 1 as defined in Table 2)

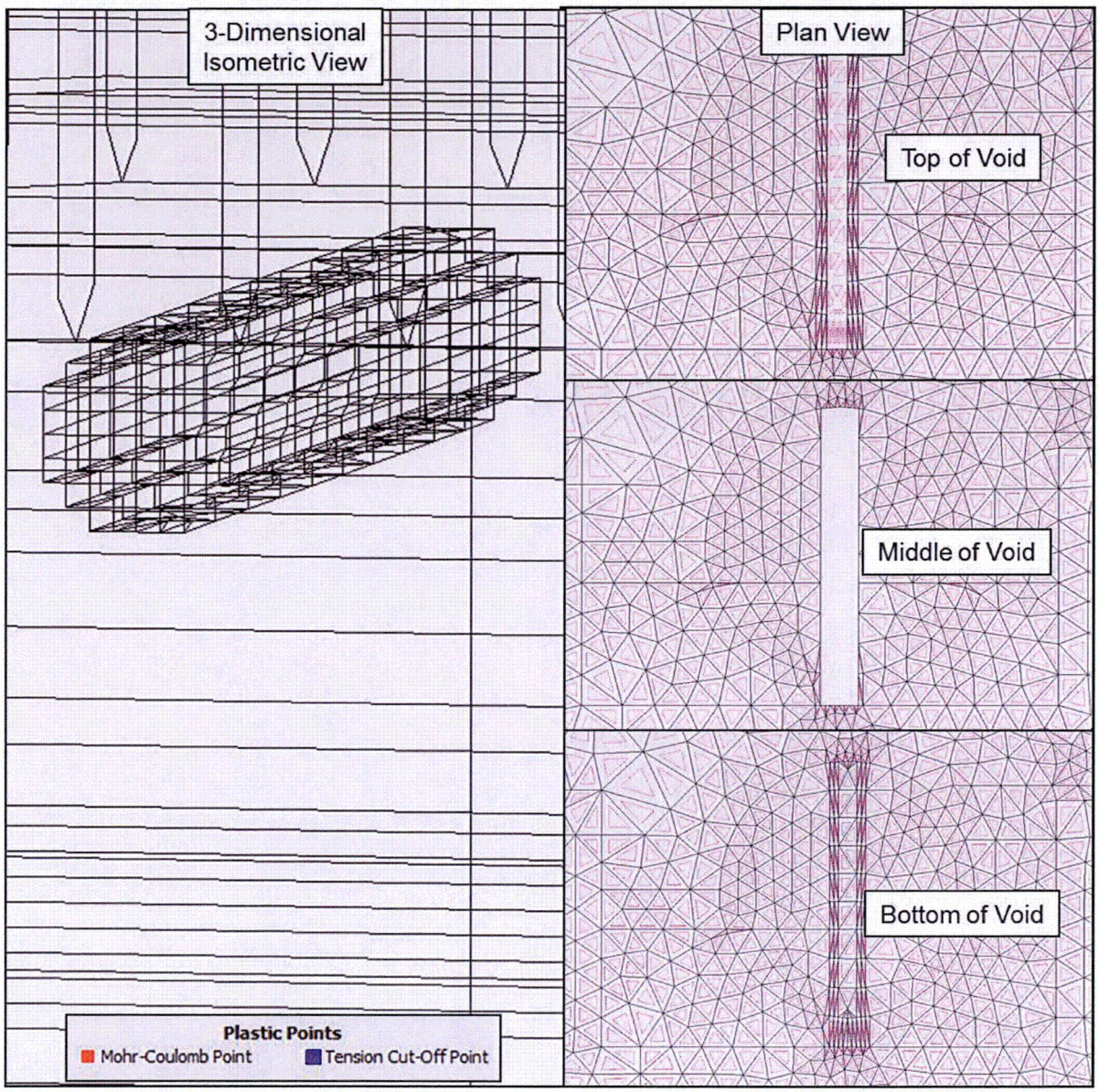
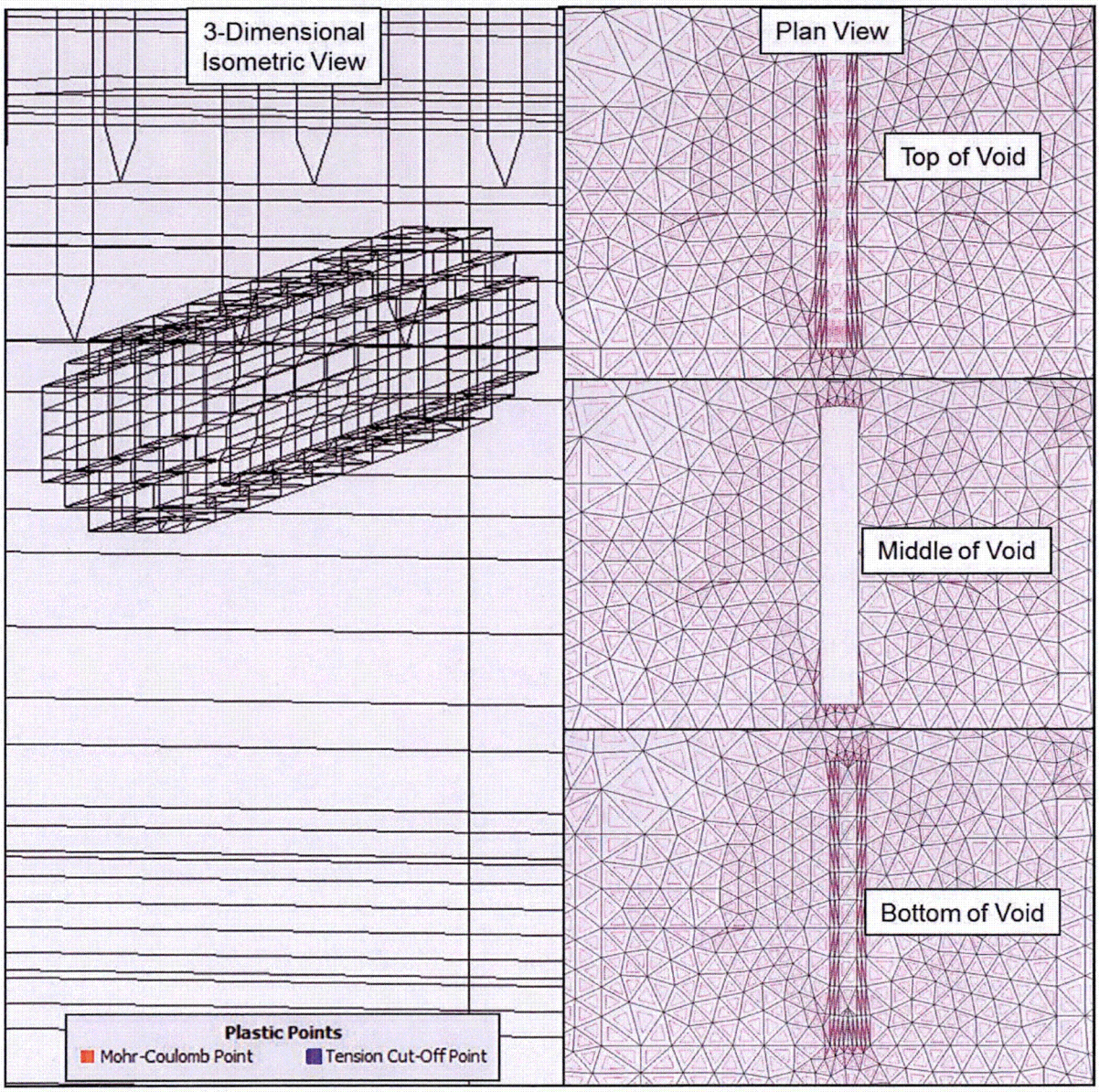
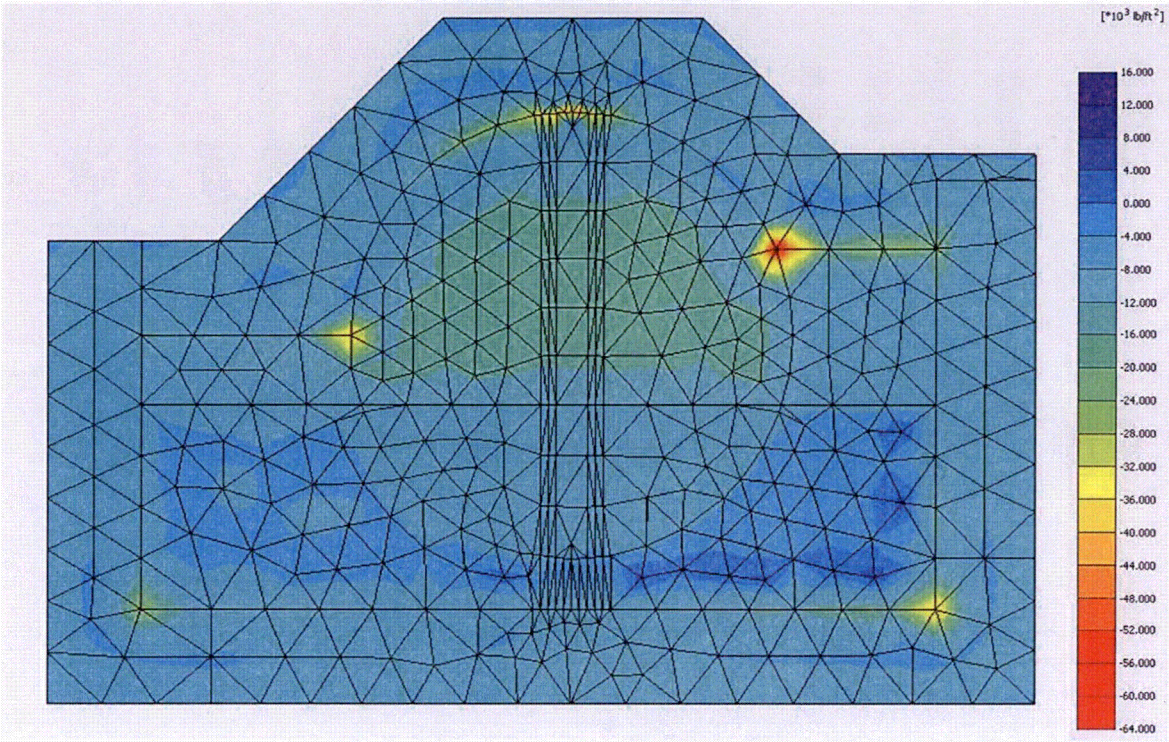


Figure 16 Plastic Points PLAXIS 3D
(Static and Pseudo-Dynamic Loading, Multiplier of 2 as defined in Table 2)



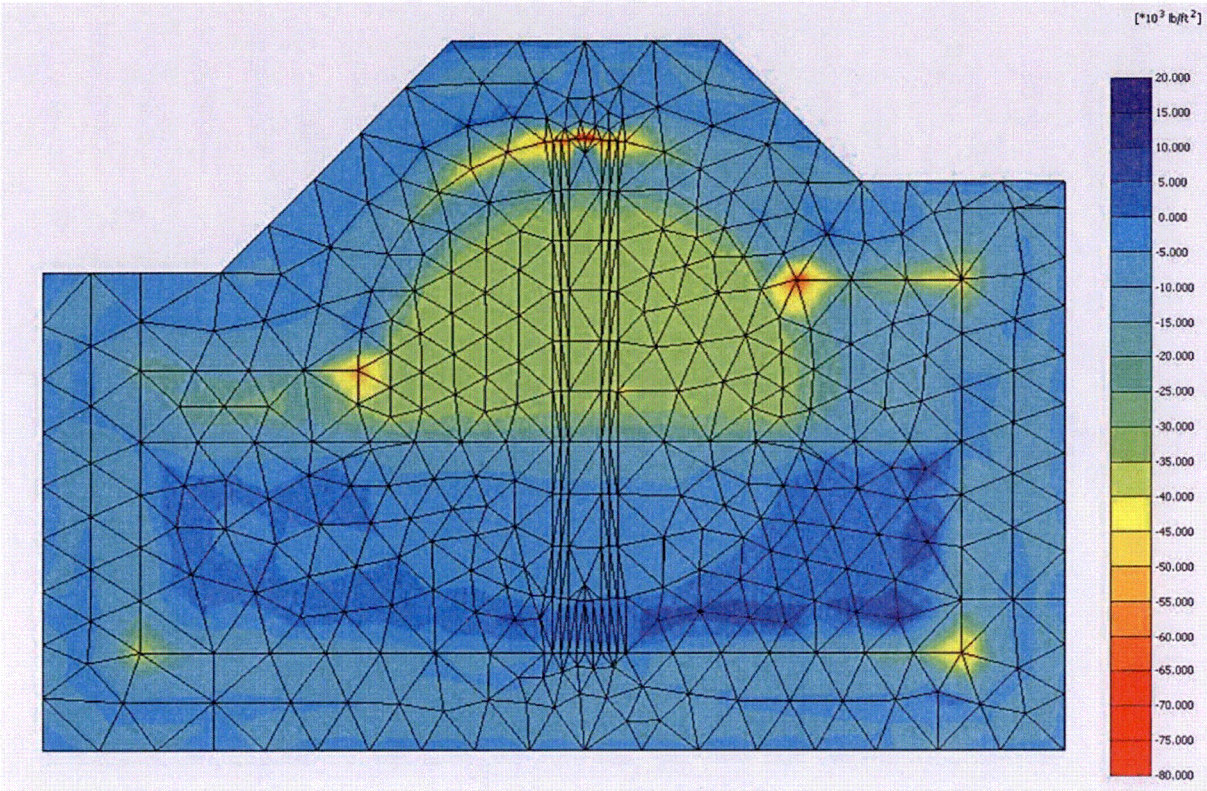
Effective vertical stresses for pseudo-dynamic loading (multiplier of 1) are shown for the concrete fill in Figure 17. Effective vertical stresses for pseudo-dynamic loading (multiplier of 2) are shown for the concrete fill in Figure 18. The maximum compressive stresses are provided in Table 4 and are much less than the ultimate bearing capacity of the concrete. The maximum compressive stresses provided in Table 4 conservatively represent local maximums for one element and one node only; these stresses are not averaged over multiple elements to distribute the stresses.

Figure 17 Effective Vertical Stresses on Concrete Fill (Static and Pseudo-Dynamic Loading, Multiplier of 1 as defined in Table 2)



Note: Negative effective stresses represent compression, while positive effective stresses represent tension.

Figure 18 Effective Vertical Stresses on Concrete Fill (Static and Pseudo-Dynamic Loading, Multiplier of 2 as defined in Table 2)



Note: Negative effective stresses represent compression, while positive effective stresses represent tension.

Table 4
Effective Vertical Stress on Concrete Fill (Pseudo-Dynamic Loading)

Pseudo-Dynamic Loading Condition	Maximum Compressive Stress (ksf)	Ultimate Bearing Capacity of Concrete (ksf)
Multiplier of 1	61	184
Multiplier of 2	77	

As indicated by the results shown in Figures 15 through 18 and in Table 4:

- No plastic points or tension cut-off points are observed during the pseudo-dynamic loading conditions,
- The effective vertical compressive stresses are smaller than the ultimate bearing capacity of the concrete, and
- The tensile capacity of the concrete fill is not reached.

In summary, the void size constrained by the grouting program has been demonstrated to not be critical to the pseudo-dynamic stability of safety related structures. In other words, subsurface collapse is not anticipated under the combination of seismic and static Nuclear Island loads.

Part 2)

As discussed in Part 1 of this response, the void size constrained by the grouting program for the Extended Grouted Zone has been evaluated and is not critical to safety related structures. Therefore, any voids smaller than those constrained by the grouting program are also not critical to safety related structures.

Part 3)

As discussed in Part 1 of this response, microgravity methods can only detect large voids at depth. Therefore, the grouting program will be utilized to constrain the void sizes. Within the diaphragm wall, primary and secondary grout boreholes will be drilled and pressure grouted between El. –35 feet and El. –60 feet (the Grouted Zone), and the grout closure criteria will be met. Therefore, potential voids within this zone are filled. Further, primary grout boreholes will be drilled and pressure grouted down to El. –110 feet (the Extended Grouted Zone), therefore constraining the size of potential voids to 20 feet.

Part 4)

The ITAAC set of actions and criteria established for this foundation construction (grouting) activity are necessary and sufficient to provide reasonable assurance that, when met, the stability of Category I structure foundations is in conformance with the combined license. Table 5, below, provides the ITAAC that when successfully executed will result in any remaining voids in the Grouted Zone being structurally insignificant and any remaining voids in the Extended Zone having a maximum equivalent spherical diameter of equal to or less than 20 feet. This is accomplished through drilling and pressure grouting of grout boreholes in accordance with the grout closure criteria and grout borehole spacing.

Table 5 (Sheet 1 of 2)
ITAAC for Seismic Category I Structure Foundation Grouting (1)

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
Inside the region defined by the diaphragm walls, drilling and pressure grouting is performed. The grout closure criteria, when used in conjunction with the specified grout borehole spacing: • Will result in any remaining voids between El. –35 ± 2 feet and El. –60 ± 2 feet (the Grouted Zone) being structurally insignificant, which is accomplished through drilling and pressure grouting of primary and secondary grout boreholes, and, if necessary, as indicated by site data, tertiary and quaternary grout boreholes; and	i. Testing and analysis will be performed through a grout test program to define grout closure criteria for both the Grouted Zone and Extended Grouted Zone, as follows: • For both the Grouted Zone and Extended Grouted Zone, the grout test program will identify and define grout closure criteria for grout consistency to ensure the grout flows into and fills potential voids in the vicinity of each grout borehole, and • For the Grouted Zone, the grout test program will identify and define grout closure criteria for identifying when each grout borehole has been filled and pressurized with grout and filling may cease or tertiary or quaternary grout boreholes are necessary, and	i. The grout closure criteria, when used in conjunction with the specified borehole spacing, will ensure that any voids remaining in the Grouted Zone are structurally insignificant and ensure that any voids remaining in the Extended Grouted Zone are equal to or less than 20 feet ii. Grout closure criteria as established in the grout test program are met inside the region defined by the diaphragm walls and the grout boreholes meet the following requirements: • For the Grouted and Extended Grouted Zones, primary grout boreholes are drilled throughout the entire interior region defined by the diaphragm walls and with spacing of less than or equal to 20 feet on center at the ground surface,

Table 5 (Sheet 2 of 2)
ITAAC for Seismic Category I Structure Foundation Grouting (1)

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
• Will result in any remaining voids between El. –60 ± 2 feet and El. –110 ± 2 feet (the Extended Grouted Zone) having a maximum equivalent spherical diameter of equal to or less than 20 feet, which is accomplished through drilling and pressure grouting of primary grout boreholes and, if necessary, as indicated by site data, secondary grout boreholes.	• For the Extended Grouted zone, the grout test program will identify and define grout closure criteria for identifying when each grout borehole has been filled and pressurized with grout and filling may cease or secondary grout boreholes are necessary. ii. Inspections and analysis will be performed of the as-built locations, depth and spacing of all grout boreholes, both with respect to the Grouted Zone and the Extended Grouted Zone, and the grout data associated with each grout borehole and zone.	• For the Grouted Zone, secondary grout boreholes are drilled throughout the entire interior region defined by the diaphragm walls and are offset from primary grout boreholes such that a secondary grout borehole is at the center of the square formed by four adjacent primary grout boreholes at the ground surface, and • Each additional grout borehole (tertiary or quaternary) drilled to meet grout closure criteria for the Grouted Zone is located based on a documented engineering evaluation consistent with the grout closure criteria • Each additional grout borehole (secondary) drilled to meet grout closure criteria for the Extended Grouted Zone is located based on a documented engineering evaluation consistent with the grout closure criteria.

Note:

(1) All elevations are presented in the North American Vertical Datum of 1988 (NAVD88).

REFERENCES:

1. FPL Letter L-2015-047 to NRC dated February 19, 2015, FSAR Subsection 2.5.4 Grout Test Program Description
2. FPL Letter L-2015-199 to NRC dated July 15, 2015, Response and Revised Response to NRC Request for Additional Information Letter No. 082 (eRAI 7811) SRP Section 02.05.04 – Stability of Subsurface Materials and Foundations
3. American Concrete Institute, *Guide to Durable Concrete* (ACI 201.2R-08), 2008
4. Lombardi, G., *Grouting of Rock Masses*, 3rd International Conference on Grouting and Grout Treatment, March 2003.
5. U.S. Army Corps of Engineers, *Engineer Manual 1110-2-3506, Grouting Technology*, January 1984.
6. Bowles, J., *Foundation Analysis and Design*, 5th ed., McGraw-Hill Companies, Inc., New York, 1997.
7. McCormac, J., Nelson, J., *Design of Reinforced Concrete*, Seventh Edition, John Wiley & Sons, Inc., 2006.
8. American Concrete Institute, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-11, 2011.
9. Brinkgreve, R.B.J. and Swolfs, W.M., *PLAXIS 3D Foundation Version 2 Part 2: Reference Manual*, PLAXIS bv, 2007.
10. American Concrete Institute, Guide to Mass Concrete, ACI 207.1R-05, Farmington Hills, Michigan, 2006.

ASSOCIATED COLA REVISIONS:

FSAR Subsection 2.5.4.4.5.5 will be revised in a future COLA revision as follows:

2.5.4.4.5.5 Summary and Commitment

Based on geophysical site characterization data (References 286 and 320), and drilling observations as outlined in Subsection 2.5.4.1.2.1, there is no apparent indication that sinkhole hazards exist at the site. There is also no apparent evidence for the presence of underground openings within the survey area that could result in surface collapse. Large low gravity anomalies with magnitudes less than –30 μGals are only detected outside the power block areas, primarily in areas associated with surface depressions containing vegetation. Once the effects of variations in muck thickness are removed from the residual gravity data, all the remaining low gravity anomalies can be explained by density variations within the Miami Limestone. The results of the drilling program and borehole geophysical data (Subsections 2.5.1.2.4 and 2.5.4.1.2.1) indicate the existence of two preferential secondary porosity flow zones. The extent of rod drops integrated with the field geophysical data supports the interpretation that large voids are absent beneath the footprints of the Units 6 & 7 nuclear islands.

However, considering the uncertainties related to resolution in the geophysical data at depth and away from survey lines, the subsurface grouting program ~~will be~~**is** considered in the determination of constrained void sizes. The zone between El. –35 feet and El. –60 feet within the diaphragm walls **(the Grouted Zone)** ~~will be~~**is** grouted according to the **grout** closure criteria **(for individual grout stages and for completed areas of the Grouted and Extended Grouted Zones)** that ~~will be~~**are** developed as part of the grout test program (determined from the results of water pressure tests, evaluation of available boring data, and the target residual permeability of the ~~grouted zone~~**Grouted Zone**). This grouting ~~will~~ **results in any remaining voids having an insignificant impact on the stability of Category I structure foundations (or are structurally insignificant)**~~ensure that potential voids in this zone are filled~~. **The void size defined as structurally insignificant is determined in the grout test program.** In addition, for the zone between El. –60 feet and El. –110 feet within the diaphragm walls **(the Extended Grouted Zone)**, grouting ~~will be~~**is** performed in every primary grout borehole. Primary grout holes ~~will be~~**are** spaced less than or equal to 20 feet on center (Figure 2CC-239). This configuration ~~is expected to constrain~~**constrains** the maximum undetected void size to approximately 20 feet. **The ITAAC set of actions and criteria established for this foundation construction (grouting) activity are necessary and sufficient to provide reasonable assurance that, when met, the stability of Category I structure foundations is in conformance with the combined license. Specifically, successful grouting ITAAC execution results in any remaining voids in the Grouted Zone being structurally insignificant, and any remaining voids in the Extended Grouted Zone having a maximum equivalent spherical diameter of equal to or less than 20 feet. The grouting ITAAC are provided in Part 10 of the COL Application in Appendix B Table 3.8-6 and are discussed in Subsection 14.3.3.6.**

The third and fourth paragraphs of FSAR Subsection 2.5.4.5.1.2 will be revised in a future COLA revision as follows:`

2.5.4.5.1.2 Power Block and Site Grade Raising

Structural fill consisting of excavated fill material is placed around but not below any nuclear island structure. Replacement material below the nuclear islands consists of concrete fill **and**~~. The selection of concrete fill mix design is made at project detailed design. A~~ **a** mix is selected that achieves the mechanical properties used for the design analyses. The compressive strength of 1.5 ksi is ~~estimated~~**chosen** for concrete fill. To ensure that the compressive strength is equal to or greater than this value, concrete test cylinders are made in the field and tested according to ACI 311.5 (Reference 324). The selection of the mix considers the strength requirements as well as the durability requirements to prevent potential sulfate attack. **The ITAAC set of actions and criteria established for this foundation construction (concrete fill compressive strength and methods to control thermal cracking) activity are necessary and sufficient to provide reasonable assurance that, when met, the stability of Category I structure foundations is in conformance with the combined license. Specifically, successful concrete fill ITAAC execution ensures that the concrete fill placed**

underneath Seismic Category I structures meets the specifications in ACI 207.1R-05. The concrete fill ITAAC are provided in Part 10 of the COL Application in Appendix B Table 3.8-5 and are discussed in Subsection 14.3.3.5.

The concrete fill exposure to sulfate attack from groundwater is classified as Class 2 exposure according to Reference 323. Recommendations for improving sulfate resistance are provided in the ACI Guide to Durable Concrete, ACI 201.2R-08 (Reference 323). For the first lift of concrete (bottom lift), the requirements in Table 6.3 of Reference 323 for water-cementitious material ratio and type of cementitious materials are followed in order to provide resistance to sulfate attack. The minimum thickness of the first lift of concrete fill is 2.5 feet. The concrete mix for the first lift contains a maximum water-cementitious material ratio by mass of 0.45, and a sulfate resisting Type V cement or equivalent as defined in Sections 6.2.5, 6.2.7, and 6.2.9 of the ACI Guide to Durable Concrete, ACI 201.2R-08 (Reference 323). In addition, Type V cement or equivalent according to ACI 201.2R-08 (Reference 323) is used for all the lifts for additional protection. Delivery tickets are prepared according to ACI 311.5 (Reference 324) and inspected to ensure that the water-cementitious material ratio and the type of cementitious materials for the first lift meet durability requirements in Reference 323 for Class 2 sulfate exposure. **The ITAAC set of actions and criteria established for this foundation construction (concrete fill sulfate attack resistance) activity are necessary and sufficient to provide reasonable assurance that, when met, the stability of Category I structure foundations is in conformance with the combined license. Specifically, successful concrete fill ITAAC execution ensures that the first lift of concrete fill placed underneath Seismic Category I structures meets the ACI 201.2R-08 durability requirements. The concrete fill ITAAC are provided in Part 10 of the COL Application in Appendix B Table 3.8-5 and are discussed in Subsection 14.3.3.5.**

FSAR Subsection 2.5.4.6.2 will be revised in a future COLA revision as follows:

2.5.4.6.2 Construction Dewatering

The excavation for each new unit will be surrounded by a reinforced concrete diaphragm wall that will act as a cut-off for horizontal groundwater flow into the excavation. Conceptual plans indicate each excavation will have dimensions of approximately 210 feet by 310 feet. The planned bottom of the wall is at El. –60 feet, i.e., just below a layer of limestone situated between the Key Largo Limestone (Subsection 2.5.4.2.1.3) and the Fort Thompson Formation (Subsection 2.5.4.2.1.2.4) that is considerably less permeable than either of these strata. This is referred to as the Freshwater Limestone in Appendix 2BB and Appendix 2CC. The layer has a lower permeability and thus reduces the amount of vertical inflow into the bottom of the excavation during dewatering.

The existing groundwater elevation in the power block areas is dependent on tidal variations, but averages close to El. 0 feet. The base of the excavation for the nuclear island is approximately El. –35 feet. Thus, temporary construction dewatering is needed down to at least El. –35 feet. The pumping test program described in Subsection 2.4.12.1.4 resulted in the development of estimates of the hydraulic conductivity of the

Freshwater Limestone and the underlying Fort Thompson Formation. Freshwater Limestone used in the groundwater model described in Appendix 2CC is assumed to have a vertical hydraulic conductivity of approximately 2.3E-06 cm/sec, compared to approximately 1.7E-01 cm/sec for the Fort Thompson Formation. In the groundwater model, the Freshwater Limestone is assumed to be absent if the available information (from borings, etc.) indicates a thickness of less than 1.5 feet.

Various geologic features can provide potential pathways for water flow such as: zones of secondary porosity, fractures, bedding planes, and voids. As discussed in Appendix 2.5AA, zones of secondary porosity contain vugs on the order of centimeter scale. Fracturing and jointing at the site is widely spaced except under the vegetated depressions and drainages, where the Miami Limestone, Key Largo Limestone, and Fort Thompson formations are slightly to moderately fractured as observed within the inclined borings of the supplemental investigation (Reference 290). **Based on the field data described above, a void size is defined as equal to or greater than 0.5 feet.**

Within the slightly to moderately fractured zones, openness of discontinuities varies from tight (no visible separation) to moderately wide (0.03 to 0.1 feet), averaging slightly open (less than 0.003 feet). As discussed in Subsection 2.5.4.1.2.1, the largest potential voids or sediment infills that were found are limited in size and extent. As discussed in Subsection 2.5.4.4.5.5, **Part 10 of the COL Application in Appendix B Table 3.8-6 provides an ITAAC that when successfully executed will result in** ~~potential~~**any remaining** voids ~~in the grouted zone~~**between El. –35 feet and El. –60 feet** ~~will be~~ **being structurally insignificant**~~grouted~~ and **any remaining** ~~the maximum equivalent spherical diameter of potential~~ voids between El. –60 feet and El. –110 feet **having a maximum equivalent spherical diameter of equal to or less than** ~~will be constrained to~~ 20 feet**, which is accomplished** by the grout program. **The elevation range from El. –35 feet to El. –60 feet is called the "Grouted Zone."** The elevation range from El. –60 feet to El. –110 feet is called "the ~~extended zone~~**Extended Grouted Zone**." ~~Based on the field data described above, a void size is defined as equal to or greater than 0.5 feet.~~

For construction-related groundwater control, a grouted zone or "plug" will be constructed via grout injections into the rock mass between the bottom of the excavation at approximately El. –35 feet and the bottom of the diaphragm wall at approximately El. –60 feet. In general, **for the Grouted Zone,** grouting will be performed in a series of split spaced borings starting with primary order grout boreholes, and continuing through secondary order grout boreholes at a minimum. **The term "grout borehole" refers to holes drilled for grouting operations and does not necessarily mean that physical samples will be obtained and geologically described. The term "verification boring" refers to holes drilled and water pressure tested where physical samples are obtained to physically and visually assess the suitability of grouting parameters as well as geologically describing the cores. Verification borings not meeting the residual hydraulic conductivity criteria will be pressure grouted. Verification borings meeting the residual hydraulic conductivity criteria will be backfilled with grout. For** ~~In the~~ primary grout boreholes **in the Grouted Zone**, individual grout stages will ~~initially~~ be grouted to **grout**

stage ~~"absolute refusal"~~ closure criteria. **The grout stage closure criteria will be developed based on results of the grout test program.** ~~Absolute refusal will be defined as zero (no measureable) flow of grout into the rock formation while the injection pressure is maintained equal to the target pressure for the stage for a duration of five minutes.~~

~~After enough (not more than eight adjacent) of the primary grout boreholes have been grouted using these criteria, the closure criteria may be adjusted based on grout takes, the results of water pressure tests, evaluation of available boring data, and the target residual permeability of the grouted zone.~~ Upon completion of the grout stages in the primary grout boreholes, secondary grout boreholes will be drilled and grouted to the ~~adjusted~~**grout stage** closure criteria. The water pressure test results **from verification borings** and grout takes from the primary and secondary grout boreholes will be evaluated to determine the need for tertiary and higher order grout boreholes. Tertiary grout boreholes are likely to be required in some areas. ~~Quarternary~~**Quaternary** grout boreholes are anticipated to be minimal but may be needed at some locations where excessive grout take occurs in higher order grout boreholes. Grouting parameters will be measured in real-time including injection pressures, rate of injection, ~~Apparent Lugeon value (hydraulic conductivity),~~ and total volume of grout. When the grout takes have been reduced, the residual hydraulic conductivity of the grout mass will be determined via water pressure tests performed in cored verification borings in the area.

An area of the ~~grouted zone~~**Grouted Zone** will be accepted as complete when the results of verification borings indicate that the residual hydraulic conductivity of the rock mass is equal to or below the target residual hydraulic conductivity.

As discussed in Subsection 2.5.4.4.5.5, primary grout boreholes will be extended to El. –110 feet (i.e., just above the interface between the Fort Thompson and Upper Tamiami formations) in order to constrain the maximum undetected spherical void size to approximately 20 feet. **The water pressure test results from verification borings and grout takes from the primary grout boreholes will be evaluated to determine the need for secondary grout boreholes. An area of the Extended Grouted Zone will be accepted as complete when the results of verification borings indicate that the residual hydraulic conductivity of the rock mass is equal to or below the target residual hydraulic conductivity. The Extended Grouted Zone will have a different residual hydraulic conductivity requirement than the Grouted Zone, as the purpose of grouting the Extended Grouted Zone is to constrain maximum undetected void size, not to reduce the hydraulic conductivity of the rock mass.**

The groundwater model simulation (Appendix 2CC) assumes hydraulic conductivity of the grout plug is 1.0E-04 cm/sec. The corresponding predicted groundwater extraction rate is 96 gpm per unit. In addition to using this value of hydraulic conductivity, a series of sensitivity analyses using a range of hydraulic conductivities (1.0E-03, 1.0E-05 and 1.0E-06 cm/sec) is conducted to determine the feasible range of dewatering discharge rates, which range from approximately 1000 to 1 gpm per unit. These values demonstrate that grouting can significantly reduce the quantity of discharge water generated during excavation dewatering activities.

FSAR Subsection 2.5.4.6.2.1 will be revised in a future COLA revision as follows:

2.5.4.6.2.1 Grout Test Program

A grout test program will be performed to validate the grout design and grouting techniques, ~~and~~to determine the approximate grout takes for the Key Largo Limestone and Fort Thompson formations**, and to determine the grout closure criteria for individual grout stages and for completed areas of the Grouted and Extended Grouted Zones**.

Grouting will be performed to facilitate construction dewatering and is not classified as safety-related. Mix design, material control, laboratory testing, grout placement, and field testing will be performed under a quality program.

The layout for the grout test program will be selected to resemble the planned construction grouting configuration. Grout borehole spacing will be set with regard to the spacing of the dominant geologic features and the construction grouting configuration. Grout borehole orientations and inclinations will be selected to promote intersections with the dominant fractures and bedding features in the area of the work. Since the fractures in the Key Largo Limestone and Fort Thompson formations range from vertical and subvertical to around 40 degree dip, it is anticipated that the inclination of the grout boreholes will be adjusted to best intercept the dominant features in the treatment area.

It is anticipated that on the order of ten primary grout boreholes will be drilled for the grout test program, with a spacing of approximately 20 feet. It is anticipated that on the order of five secondary grout boreholes will be drilled for the grout test program, and will be offset from primary grout boreholes such that a secondary grout borehole is at the center of the square formed by four adjacent primary grout boreholes. Verification borings will be drilled at various locations within the grouted area to measure residual hydraulic conductivity of the rock mass and to physically and visually assess the suitability of grouting parameters. It is anticipated that five verification borings will be drilled for the grout test program.

The grout test program will be used to optimize and finalize the grouting and dewatering specifications, including:

- Spacing for primary and secondary grout boreholes. Primary grout boreholes will be spaced less than or equal to 20 feet on center. Spacing of primary and secondary grout boreholes may be reduced based on results from the grout test program.
- Suitability of the formation for grouting via downstages, upstages, or a combination of upstages and downstages. Upstage grouting is a method whereby packers or expansion plugs block off preselected portions of the grout boreholes in ascending stages while those portions are being grouted. In this method, grout boreholes are drilled to their full depth, pressure tested, and grouted in successive stages from the bottom up. Alternatively, downstage grouting is the grouting of progressively deeper zones in stages, with the deeper zones accessed by drilling through previously injected grout. Effectiveness of the staging method will be determined by **grout** borehole conditions during drilling including **grout** borehole instability during or after

drilling, loss of circulation of drill fluid, difficulties setting packers in **grout** boreholes during ~~water pressure testing and~~ grouting, and by the level of improvement of **grout** borehole conditions after grouting has been performed in an area.

- Inclination of grout boreholes (vertical or inclined). As described above, **grout borehole** ~~boring~~ orientations and inclinations will be selected to promote intersections with the dominant fractures and bedding features in the area of the work. It is anticipated that the inclination of the grout boreholes will be adjusted during the grout test program to best intercept the dominant features.
- Effective grout mixes using locally available materials and water sources. Multiple grout mixes will be identified for high and low mobility grout. Grout mixes will be tested to assess their mobility, stability, and durability. Different grout mixes will be considered for the ~~grouted zone~~**Grouted Zone** and the ~~extended zone~~**Extended Grouted Zone**. As shown in Table 2.4.12-211, the sulfate values measured from 24 water samples range from 2280 ppm to 4400 ppm, resulting in a median value of about 3800 ppm, or close to 0.4 percent by weight. **This classifies the concrete exposure to sulfate attack as Class 2 exposure according to ACI 201.2R-08 (Reference 323).** The **amount of** grout **pumped into** ~~within~~ potential voids is **expected to be minimal (no physical evidence indicating large voids), and variable across the site.**~~not expected to be a uniform material in terms of strength-stiffness properties. Therefore, potential cracking within the grout material due to sulfate attack will not significantly alter the mechanical response of the grout-rock mass. In addition, stability~~ **Stability** analyses do not consider any increase in strength or stiffness due to the presence of the grout.
- Drilling and flushing of grout boreholes. The grout boreholes will be advanced using water flushed, rotary, or rotary percussive drilling. The primary grout boreholes in an area will be drilled~~, water pressure tested,~~ and grouted before the drilling proceeds on the secondary grout boreholes. ~~Primary and secondary grout~~**Grout** borehole rock drilling will be monitored for penetration rate, down thrust pressure, rod torque, drilling fluid pressure and flow. Rock mass data will be collected using a drilling parameter recorder.
- Injection rates and pressure. Grouting pressures will be selected using **the grouting intensity method pressure-volume curves from Lombardi (Reference 328)** ~~a~~**in** combination ~~of best practices~~**with more traditional industry practice** (e.g., Reference 327 uses 0.5 psi per foot of overburden and 1.0 psi per foot of rock)~~ and experience from similar projects~~. Injection rates will be dictated by the ability to reach and maintain the target pressure. Injection volume limits will be optimized in the test program by **use of the pressure-volume curves and** evaluating the grout travel distance. Additionally, **target pressures will be evaluated from the perspective of a potential hydraulic fracture in the grouted rock and** ~~a rock pressure capacity test can be performed to determine~~ the peak allowable grouting pressure **will be established** for the site.

- Grouting conditions will be evaluated by using computer controlled real time monitoring of grout injected volumes, injection pressures, injection flow rates, ~~Apparent Lugeon values,~~grout mix changes and automatic recording of data.
- **For primary grout boreholes in the Grouted Zone, initially**~~Initially, closure criteria for~~ individual grout stages will be **grouted to "absolute refusal" grout stage closure criteria. Absolute** ~~based on absolute~~ refusal **for the stage will be defined as zero (no measureable) flow of grout into the rock formation with the injection pressure**~~, or a time interval (approximately 5 minutes) where the grout is~~ maintained at the target pressure **for the target duration of time**~~and no measurable flow is occurring~~. **After enough (not more than eight adjacent) of the primary grout boreholes have been grouted using these criteria, the grout stage closure** ~~Closure~~criteria **may be adjusted based on grout takes, the results of water pressure tests from verification borings, evaluation of available boring data, and the target residual permeability.**~~ will be modified for efficiency as the grout test program progresses.~~ If the grout take for a stage is significant, closure will be achieved by using progressively thicker mixes that have reduced mobility. If necessary to reach closure, grouting may be stopped temporarily to allow grout to set and then resumed.
- Verification borings will be drilled and cored for the performance of water pressure tests to measure the residual hydraulic conductivity of the rock mass and to physically and visually assess the suitability of grout borehole spacing and inclination. Acceptance criteria for a completed area of the ~~grouted zone~~**Grouted Zone** will be based on water pressure testing performed in verification **borings**~~boreholes~~. **For example, a**~~A~~ target area residual hydraulic conductivity of 10 Lugeons or less is **generally** reasonable. The acceptance criteria will be confirmed to be adequate for the site based on the results from the grout test program. The ~~extended zone~~**Extended Grouted Zone** will ~~not~~ have a **different** residual hydraulic conductivity requirement, as the purpose of grouting this zone is to constrain maximum undetected void size.

The second paragraph of FSAR Subsection 2.5.4.10.8 will be revised in a future COLA revision as follows:

2.5.4.10.8 Stability of Category I Structures Considering Postulated Voids in Subsurface

As discussed in Subsection 2.5.4.4.5.5, the grouting program ~~will be~~ **is** utilized to constrain void sizes. The zone between El. –35 feet and El. –60 feet within the diaphragm walls **(the Grouted Zone)** ~~will be~~ **is** grouted using a multi-stage grouting program; this ~~will~~ **results in any remaining voids in this zone being structurally insignificant**~~ensure that potential voids in this zone are grouted~~. For the zone between El. –60 feet and El.–110 feet within the diaphragm walls **(the Extended Grouted Zone)**, grouting ~~will be~~ **is** conducted in every primary grout borehole, constraining the maximum undetected void size to approximately 20 feet.

FSAR Subsection 2.5.4.12 will be revised in a future COLA revision as follows

2.5.4.12 Techniques to Improve Subsurface Conditions

~~Given the depths of structure foundations and the subsurface conditions that occur at those depths, as shown in part on Figures 2.5.4-221 and 2.5.4-222, special ground improvement measures are not warranted.~~ Ground treatment is limited to over-excavation of unsuitable materials, such as zones of less competent materials occurring at foundation subgrades, and their replacement with lean concrete fill. Groundwater control **(accomplished by grouting)** is required as part of this over-excavation as described in Subsections 2.5.4.5 and 2.5.4.6. **Additionally, grouting is used to constrain the size of potential voids, as described in Subsection 2.5.4.4.5.5.**

Over-excavation of approximately 21 feet at the reactor/auxiliary building is designed to replace soils and weak rock that are not adequate to directly support the high foundation loads of these structures, with the required FOS. For all affected structures, compacted limerock fill and lean concrete fill are placed according to engineering specifications and quality control/quality assurance testing procedures established during detailed design phase.

According to ACI 207 (Reference 281), the lean concrete fill under the nuclear island is defined as mass concrete. A thermal control plan considering the geometry of the fill concrete, the proposed 1,500 psi strength, total volume of fill concrete placement, and rate of concrete production, ~~will be~~**is** prepared to minimize thermal cracking in accordance with ACI 207 guidelines.

As described in Subsection 2.5.4.4.5.5, the zone between El. –35 feet and El. –60 feet (the Grouted Zone) is grouted according to the grout closure criteria that are developed as part of the grout test program, therefore resulting in any remaining voids in this zone being structurally insignificant. In addition, for the zone between El. –60 feet and El. –110 feet (the Extended Grouted Zone), grouting is performed in every primary grout borehole. Primary grout boreholes are spaced less than or equal to 20 feet on center, therefore constraining the maximum undetected void size to approximately 20 feet.

Across the entire plant area, the muck of Stratum 1 is removed and replaced with compacted limerock fill as described in Subsection 2.5.4.5.1.1.

The following reference will be added to Subsection 2.5.4.13 in a future revision of the COLA:

328. Lombardi, G., *Grouting of Rock Masses*, 3rd International Conference on Grouting and Grout Treatment, March 2003.

FSAR Subsection 14.3.3.5 will be revised in a future COLA revision as follows:

14.3.3.5 Concrete Fill ITAAC

The ITAAC set of actions and criteria established for this foundation construction (concrete fill) activity are necessary and sufficient to provide reasonable assurance that, when met, the stability of Category I structure foundations is in conformance with the combined license. Subsection 2.5.4.5 discusses, in part, the excavations, backfill (including cementitious construction material) and earthwork analyses for Seismic Category I structures. The objective of this concrete fill ITAAC is to ensure reliable performance of the foundation bearing material over the life of the plant. Specifically, ~~proper~~**successful concrete fill** ITAAC **execution ensures that**~~are specified to ensure~~ the first lift of concrete fill material is resistant to sulfate attack. By verifying water-cementitious material ratio and cement type, this ITAAC provides a method to confirm that sulfate-resistant properties of the fill material are achieved.

~~Additionally, the~~**Successful concrete fill** ITAAC **execution also ensures**~~have been developed to ensure~~ that the static and dynamic properties of the material ~~will be~~**are** the same as, or better than the design parameters. In general, by testing the mean 28-day compressive strength of cementitious construction material, this ITAAC provides a method to confirm that the properties (static and dynamic) of said material are met prior to the construction of the Seismic Category I structure.

FSAR Subsection 14.3.3.6 will be added in a future COLA revision as follows:

14.3.3.6 ITAAC for Category I Structure Foundation Grouting

The ITAAC set of actions and criteria established for this foundation construction (grouting) activity are necessary and sufficient to provide reasonable assurance that, when met, the stability of Category I structure foundations is in conformance with the combined license. This ITAAC ensures that the zone between El. –35 feet and El. –60 feet within the diaphragm walls (the Grouted Zone) is grouted according to the grout closure criteria that are developed as part of the grout test program. Specifically, successful grouting ITAAC execution results in any remaining voids in the Grouted Zone being structurally insignificant. The void size defined as structurally insignificant is determined in the grout test program. In addition, for the zone between El. –60 feet and –110 feet within the diaphragm walls (the Extended Grouted Zone), grouting is performed in every primary grout borehole. Primary grout boreholes are spaced less than or equal to 20 feet on center. Specifically, successful grouting ITAAC execution results in any remaining voids in the Extended Grouted Zone having a maximum equivalent spherical diameter of equal to or less than 20 feet. By verifying that the grout closure criteria of each zone are met and the as-built locations of the grout boreholes, this ITAAC provides a method to confirm that any remaining voids in the Grouted Zone are structurally insignificant and that the maximum equivalent spherical diameter of remaining voids in the Extended Grouted Zone is equal to or less than 20 feet.

Table 3.8-6 (Sheet 1 of 2)
Seismic Category I Structure Foundation Grouting (1)

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
Inside the region defined by the diaphragm walls, drilling and pressure grouting is performed. The grout closure criteria, when used in conjunction with the specified grout borehole spacing: • Will result in any remaining voids between El. –35 ± 2 feet and El. –60 ± 2 feet (the Grouted Zone) being structurally insignificant, which is accomplished through drilling and pressure grouting of primary and secondary grout boreholes, and, if necessary, as indicated by site data, tertiary and quaternary grout boreholes; and	i. Testing and analysis will be performed through a grout test program to define grout closure criteria for both the Grouted Zone and Extended Grouted Zone, as follows: • For both the Grouted Zone and Extended Grouted Zone, the grout test program will identify and define grout closure criteria for grout consistency to ensure the grout flows into and fills potential voids in the vicinity of each grout borehole, and • For the Grouted Zone, the grout test program will identify and define grout closure criteria for identifying when each grout borehole has been filled and pressurized with grout and filling may cease or tertiary or quaternary grout boreholes are necessary, and	i. The grout closure criteria, when used in conjunction with the specified borehole spacing, will ensure that any voids remaining in the Grouted Zone are structurally insignificant and ensure that any voids remaining in the Extended Grouted Zone are equal to or less than 20 feet ii. Grout closure criteria as established in the grout test program are met inside the region defined by the diaphragm walls and the grout boreholes meet the following requirements: • For the Grouted and Extended Grouted Zones, primary grout boreholes are drilled throughout the entire interior region defined by the diaphragm walls and with spacing of less than or equal to 20 feet on center at the ground surface,

Table 3.8-6 (Sheet 2 of 2)
Seismic Category I Structure Foundation Grouting [1]

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
• Will result in any remaining voids between El. –60 ± 2 feet and El. –110 ± 2 feet (the Extended Grouted Zone) having a maximum equivalent spherical diameter of equal to or less than 20 feet, which is accomplished through drilling and pressure grouting of primary grout boreholes and, if necessary, as indicated by site data, secondary grout boreholes.	• For the Extended Grouted zone, the grout test program will identify and define grout closure criteria for identifying when each grout borehole has been filled and pressurized with grout and filling may cease or secondary grout boreholes are necessary. ii. Inspections and analysis will be performed of the as-built locations, depth and spacing of all grout boreholes, both with respect to the Grouted Zone and the Extended Grouted Zone, and the grout data associated with each grout borehole and zone.	• For the Grouted Zone, secondary grout boreholes are drilled throughout the entire interior region defined by the diaphragm walls and are offset from primary grout boreholes such that a secondary grout borehole is at the center of the square formed by four adjacent primary grout boreholes at the ground surface, and • Each additional grout borehole (tertiary or quaternary) drilled to meet grout closure criteria for the Grouted Zone is located based on a documented engineering evaluation consistent with the grout closure criteria • Each additional grout borehole (secondary) drilled to meet grout closure criteria for the Extended Grouted Zone is located based on a documented engineering evaluation consistent with the grout closure criteria.

Note:
[1] All elevations are presented in the North American Vertical Datum of 1988 (NAVD88)

ENCLOSURES:

None