Exhibit: List of FSAR Sections and RAI Responses

List of FSAR Sections

2.5.1.2.7	Site Engineering Geology Evaluation
2.5.4.5	Excavations and Backfill
2.5.4.12	Techniques to Improve Subsurface Conditions
2.5.4.1.3	Zones of Alteration, Irregular Weathering, or Structural Weakness
2.5.4.6.2	Construction Dewatering
2.5.4.1.2.1	Dissolution Activity
2.5.1.2.5	Geology of the Site Location

List of RAI Responses

NRC RAI #	Text of NRC RAI
02.05.04-1	Please provide a sufficiently detailed discussion to justify that the borings adequately characterize karst at depth at the site, and that the existing borehole spacing is sufficient to characterize the lateral dimension of dissolution cavities and assess their correlation and interpreted lack of connectivity between boreholes.
02.05.01-1	Please summarize the information being used as the technical basis for the dissolution rates presented, including documentation of the basis for indicating that dolomitized limestone dissolves less readily than non-dolomitized limestone, to enable an adequate assessment of karst development as a potential future geologic hazard. Include any references necessary.
02.05.01-3	The supplement states that grouting will inhibit the development of karst by preventing the flow of groundwater through the grouted zones beneath the nuclear island (Attachment 2, pg. 15 of supplement, Permeation Grouting Discussion). Please address the potential issue of how altering the groundwater flow regime by grouting could affect dissolution below and around the periphery of the grouted zone to assure that this aspect has been considered.

02.05.01-5	The supplement lists assumptions and postulations used to calculate lateral dimensions of borehole features (Attachment 2, pg. 7 of supplement, Karst Discussion - Excess Grout Takes), and states that 9.9 ft is the maximum lateral extent of dissolution cavities at depth. Considering a fracture spacing of 19 ft., if dissolution developed along two parallel fractures with this spacing, then the resulting cavity could easily exceed 9.9 ft. if the two cavities coalesced at depth. Please discuss the uncertainty involved in the estimate of a 9.9 ft. maximum lateral extent for dissolution cavities and the potential for coalescing dissolution cavities at depth.

- No natural processes that might cause uplift are active at the site.
- Unrelieved residual stresses are judged to not be a hazard to the site.
- Ground failure and differential settlement due to liquefaction are judged to not be a hazard to the site (see FSAR Subsection 2.5.4.8.5).

The only geologic hazard identified in the LNP site area is potential sulface deformation related to carbonate dissolution and subsidence related to the occurrence of karst. A discussion of the potential for surface deformation related to karst subsidence or collapse is provided in FSAR Subsection 2.5.3.8. The characterization of deeper subsurface karst features for foundation design is summarized in the following subsection and is discussed in more detail in FSAR Subsection 2.5.4.

2.5.1.2.7 Site Engineering Geology Evaluation

The engineering significance of geologic and geotechnical characteristics of features and materials, including foundation materials, are addressed in the following subsections.

2.5.1.2.7.1 Engineering Behavior of Soil and Rock

Engineering soil properties, including index properties, static and dynamic strength, and compressibility, are discussed in FSAR Subsection 2.5.4.

2.5.1.2.7.2 Zones of Alteration, Weathering, and Structural Weakness

The bedrock, which underlies the undifferentiated Quaternary sediments, is the middle Eocene-aged Avon Park Formation (FSAR Subsection 2.5.4.1). The upper portion of this formation, which consists of calcareous silts (units S2 and S3, also referred to as undifferentiated Tertiary sediment) appears to have been altered by weathering and greater degrees of dissolutioning (FSAR Subsection 2.5.1.2). No zones of structural weaknesses, such as extensive fracture zones or faults, have been identified at the LNP site. Postulated faults in the site area and vicinity have been suggested by others in literature, but more recent studies do not provide evidence of the postulated faults (FSAR Subsection 2.5.3.1). Also, regional fracture zones that are mapped in the site region do not cut across the site (FSAR Subsection 2.5.3.2.1).

Smaller subsets of these regional fractures observed in bedrock outcrops in the site area are consistent with these regional trends (FSAR Subsection 2.5.4.1.2.1.1). Bedrock outcropping was not observed in the site location, but televiewer records provide some information on fractures observed in boreholes at the site (FSAR Subsection 2.5.4.4.2.2). Additionally, as with nearly all rock formations, fractures, joints, and bedding planes exist in the Avon Park Formation. These discontinuities (vertical fractures, joints, and bedding planes) are key elements in the localization and development of karst.

2.5.1.2.7.3 Karst Features

The term “karst feature” generally includes surface sinkholes, mantled epikarst subsurface, buried ancient sinkholes (paleosinks), voids, in filled cavities, deep soil infill, and buried raveled zones. Karst features encountered within the LNP site location are expected to be associated with vertical fractures and bedding planes, and vary in lateral extent from a few centimeters to approximately 1.5 m (5 ft.), as discussed FSAR Subsection 2.5.4.1.2.1.3. Karst related features that exist in the subsurface beneath the LNP foundation will be addressed through appropriate design considerations in the LNP foundation concept as discussed in FSAR Subsection 2.5.4.

2.5.1.2.7.4 Deformational Zones

With the exception of possible paleosink features, no deformation zones have been encountered in the site characterization explorations for LNP 1 or LNP 2. Excavation mapping will be undertaken during construction to further evaluate the possibility of deformational zones (FSAR Subsection 2.5.3.8.1).

2.5.4.5 Excavations and Backfill

LNP COL 2.5-7 Soil and rock excavations will be required to construct the LNP nuclear islands on rock at a subgrade elevation of approximately -7.3 m (-24 ft.) NAVD88. This subsection describes the anticipated excavation and backfill plans for the nuclear islands, including the planned diaphragm walls, excavation extents and methods, and the properties of backfill beneath and adjacent to safety-related structures.

Construction sequencing for these activities is described in FSAR Subsections 2.5.4.12 and 3.8.5.10.

2.5.4.5.1 Diaphragm Walls and Grouting

In order to support the excavation of the nuclear islands, reinforced concrete diaphragm walls will be constructed as the boundary of the excavation limits. These excavation limits are discussed in FSAR Subsection 2.5.4.5.2 and are shown on Figures 2.5.4.5-201A, 2.5.4.5-201B, 2.5.4.5-202A, and 2.5.4.5-202B.

These diaphragm walls will be installed, prior to excavation, from the existing ground surface ranging from an approximate elevation of 12.8 to 13.1 m (42 to 43 ft.) NAVD88 at LNP 1, and approximately 12.5 to 13.1 m (41 to 43 ft.) NAVD88 at LNP 2. The diaphragm walls will serve as a temporary excavation support system to facilitate excavation to elevation -7.3 m (-24 ft.) NAVD88, and will extend in depth to elevation -16.5 m (-54 ft.) NAVD88 to support construction dewatering, as discussed in FSAR Subsection 2.5.4.6.2. Constructed approximately 9.1 m (30 ft.) into rock, the diaphragm walls will be advanced using a kelly-mounted Hydrofraise excavator, standard practice for the installation of such walls. For the portion of the diaphragm wall under the Turbine Building, Annex Building, and the Radwaste Building foundation mat, the top of the diaphragm wall will be at least 1.5 m (5 ft.) below the bottom of the respective buildings' foundation mat as shown in Figure 3.7-226.

The diaphragm walls will include seven rows of prestressed anchors, spaced as shown on Figure 2.5.4.5-203. The anchors will be inclined at 45 degrees and bonded into the limestone of the Avon Park Formation. The prestressed anchors will be placed at 3 m (10 ft.) spacing around the entire perimeter of each diaphragm wall.

For design purposes of the diaphragm walls, the concrete compressive strength is to be 4000 psi, with 1 percent reinforcement on both sides of the wall. The minimum required wall thickness is 1.1 m (3.5 ft.).

Concurrent with the installation of the diaphragm walls, a grouting program will be undertaken to form the bottom of the "bathtub" as described in FSAR Subsection 2.5.4.6.2. The grouting operation will be conducted from, at or near, the existing ground surface by drilling boreholes from the surface down to the approximate elevation of -30.2 m (-99 ft.) NAVD88. The top elevation of the

grouted zone will be at elevation -7.3 m (-24 ft.) NAVD88, resulting in a 22.9 m (75 ft.) thick grouted zone.

Grouting will generally be performed by the upstage method with pneumatic packers and a suite of grout mixes that range in viscosities from 35 seconds to over 80 seconds. Primary grout holes will be spaced on a 4.8 m (16 ft.) hexagonal pattern, and split-spaced with secondary grout holes to achieve “no take” conditions. Provisions will be in place to perform additional split-spacing to tertiary grout holes, as dictated by the performance of the production grouting. Effective grouting pressures will be limited to approximately 0.5 psi/ft of depth, monitored using a GIN curve and penetrability curve developed during the Grout Test Program. Hole spacing, grouting pressures, and acceptable grout takes will be established with the grout program. The target residual conductivity of the production grouting will be 15 Lugeons. Grouting is nonsafety-related, however it will be performed under a quality program.

2.5.4.5.1.1 Diaphragm Wall with Anchors

A diaphragm wall with prestressed tiebacks will be used as a groundwater cutoff and excavation support system to facilitate the 67-ft.-deep excavation. The analysis includes an assessment of the required diaphragm wall thickness and reinforcement, the arrangement and required number of anchors, the maximum expected anchor load for each construction stage, and the required bonding length of each anchor.

A diaphragm wall system with prestressed tiebacks is planned to enable the excavation and dewatering of the nuclear island. This continuous wall is designed as an excavation support system to facilitate the 67-ft.-deep excavation and prevent excessive groundwater from entering the excavation area.

The diaphragm wall with tiebacks was considered to be a stiff wall system, and construction-sequencing analysis using classical soil pressures was employed for the design.

An earth pressure diagram for a rigid wall (with a fixed base) consists of an apparent earth pressure on the upper section of the wall and a triangular distribution on the lower section of the wall. The earth pressures are based on the at-rest lateral earth pressure condition.

SAP2000 was used to analyze moment and shear force distribution of the continuous beam. For the reinforced concrete component design, the American Concrete Institute (ACI) 318 Ultimate Strength Design (USD) method was used.

As a design input, the groundwater level is assumed to be at ground surface behind the diaphragm wall, and 5 ft. below the excavation in front of the wall; i.e., the excavation is dewatered and there is no water pressure in front of the diaphragm wall during each stage of construction. Full hydrostatic pressure was considered behind the wall.

For each stage of construction, 2 ft. of over-excavation is considered below an anchor location. The bonding strength between grout and limestone rock is interpreted to be 200 psi (1.4 Mpa) based on published data.

The inclination of the anchors is 45 degrees, and all anchors will be keyed into competent rock. The drilled anchor holes are 6 inches in diameter.

The compressive strength of concrete is 4000 psi, and the elastic modulus of concrete 3000 ksi, calculated based on the concrete compressive strength.

The diaphragm wall includes 7 rows of prestressed anchors. To reduce the shear force and moment imposed on the wall by the earth pressure, anchors are closely spaced at the lower section of the wall, and relatively widely spaced at the upper section. The construction sequencing analysis involved eight stages of analysis, each stage considering an over-excavation of 2 ft. below the anchor location.

For the structure component design, ACI 318 USD methodology was used and a load factor of 1.2 was used for the design; allowable strength design (ASD) methodology was used for the anchor design, and a factor safety of 2.0 is used to determine the bonding length.

The concrete compressive strength is to be 4000 psi. The minimum required wall thickness is 3.5 ft., and the reinforcement ratio is to be 1 percent, reinforced on both sides (2 percent total). The embedment into rock is to elevation -54 ft. NAVD.

2.5.4.5.1.2 Permeation Grouting

Due to the high groundwater table and the documented permeability of the Avon Park Formation beneath the site, the upper 75 ft. of the Avon Park Formation will be grouted to diminish its porosity and permeability. The grouting will allow the excavation to be made in a safe and predictable manner by minimizing the upward flow of groundwater into the excavation and to aid in the resistance to uplift pressures on the excavation bottom. An uplift analysis indicated sufficient reduction of shear stresses in the grouted rock, and the computed factor of safety exceeded 1.5.

The grouting is non safety related. However, diminishing the porosity and reducing the permeability will have the beneficial effect of impeding flow through the uppermost Avon Park Formation and, therefore, minimize the potential for the initiation and/or growth of solution activity.

Although this will be an added benefit, the increase in compressive and shear strength of the Avon Park Formation has not been considered in other analyses. Bearing capacity, settlement, and site response were assessed on the basis of properties of the Avon Park Formation as measured during the site characterization program without grouting. The success of the Grout Program will be determined by the lack of groundwater intrusion during the excavation

dewatering and not the increase in density, stiffness, or strength of the Avon Park Formation.

As a design input for the determination of the grouted zone, the groundwater is conservatively considered to be at the existing ground surface (between elevation 42 ft. and elevation 43 ft. NAVD88).

As part of the construction dewatering effort, a zone beneath each proposed nuclear island will be grouted in order to achieve the following three goals:

1) Form a “bottom of the bathtub” to prevent the flow of groundwater up through the bottom of the excavation.

2) Protect the excavation base from heaving.

3) Inhibit the flow of water through porous zones in this zone beneath each nuclear island, thereby reducing the future potential for solution activity.

The top elevation of the grouted zone (elevation -24 ft. NAVD88) was based on the top of rock and defines the elevation which the RCC Bridging Mat will be founded on. The proposed thickness of this grouted zone (75 ft., to elevation -99 ft. NAVD88) was determined based on the review of site data and discussions with site geologists. For example, shear wave velocity measurements from Borings A7, I2, AD1, A8, and I3, indicate a shelf within the Avon Park Formation at approximate elevation -97 ft. NAVD88 under the North Reactor LNP 2, where shear wave velocity increases from approximately 3500 ft/sec to approximately 5000 ft/sec. Boring Logs from Borings A7, A8, A9, and A10 indicate that the Avon Park Formation, in general, becomes less weathered, has a higher recovery, and higher RQD below elevation -97 NAVD88.

A similar shelf exists under the South Reactor LNP 1 at approximately -180 feet. However, Boring Logs from Borings A14, A17, A19, and A20 indicate that the Avon Park limestone, in general, has a higher recovery and higher RQD below elevation -97 ft. Additionally, geophysical logs from A-19 and A-20 indicate a higher shear wave velocity below elevation -97 ft. NAVD88. Based on the above information, elevation -99 ft. NAVD88 has been designated as the bottom of the grouted zone resulting in a relatively large, 75-ft.-thick zone. As discussed in FSAR Subsection 2.5.4.1.2.1.1, this shelf extends at least 50 ft. in depth and is characterized as a lower-porosity zone.

Grouting 75 ft. of the Avon Park Formation beneath the RCC Bridging Mat will accomplish goals one (1), two (2), and three (3) listed above. As previously noted, no credit was taken for this grout increasing the strength or stiffness of the grouted zone.

The grout will be bounded horizontally by the diaphragm wall between the bottom of the RCC Bridging Mat (elevation -24 ft. NAVD88) and bottom of the diaphragm wall (elevation -54 ft. NAVD88). From this elevation to the bottom of the grouted

zone (elevation -99 ft. NAVD88), the grouted zone will be bounded by a grout curtain.

The Grout Program will be accomplished in two phases. Prior to the excavation of the nuclear island foundations, grout holes will be drilled from the existing ground surface to the proposed bottom of the target grouted zone (approximately 150 ft bgs). The first phase will consist of drilling and grouting on 8-ft. center-to-center spacing with a relatively low mobility grout (LMG). This LMG helps to form a perimeter to contain the second phase of grouting. The LMG grouting includes the installation of the grout curtain below the diaphragm wall. The purpose of the grout curtain is to “extend” the diaphragm wall and form a border around the grouted zone. A high mobility grout (HMG) will be drilled and grouted on split-spacing between the LMG holes. The HMG will fill in the area defined by the LMG. This is considered the second phase of the Grout Program.

State-of-the-practice computerized monitoring of all grouting will take place, including the measurement of grout take in terms of pressure and volume. A field test will be conducted prior to construction of this grouted zone to establish appropriate mixes for both the LMG and HMG and to confirm that the grout hole spacing is adequate. The 8-ft. grout hole spacing is currently based on experience in the industry. It is noted as a good starting point to be refined with a field test prior to and during construction.

2.5.4.5.1.2.1 Permeation Grouting Operation

The grouting operation will be conducted from, at or near, the existing ground surface by drilling boreholes from the surface down to the approximate elevation of -30.2 m (-99 ft.) NAVD88, and setting casing (either perforated or “tube-a-manchette” – a rubber sleeve between two packers). While uncased holes would be preferred, the existing site characterization data suggest that the holes may cave before they can be grouted; therefore, casing will be specified. The top elevation of the grouted zone will be at elevation -7.3 m (-24 ft.) NAVD88, resulting in a 22.9 m (75 ft.) thick grouted zone.

Grouting will generally be performed by the upstage method with pneumatic packers and a combination of lower mobility grout (LMG) and high mobility grout (HMG) to be established with a Grout Test Program prior to the commencement of the grouting program, as discussed in FSAR Subsection 2.5.4.5.1.2.2. Grout holes are initially spaced to achieve “no take” conditions. Hole spacing, grouting pressures, and acceptable grout takes will be established with the Grout Test Program. Grouting is non safety-related, however it will be performed under a quality program.

A grout intensity number (GIN) curve and target permeability (in Lugeons) will be used to dictate target grout pressures/volumes. The grout holes will be installed using an automated real-time monitoring system for water pressure testing and grouting, capable of computing a suite of engineering data allowing side-by-side evaluation of geology, grout mixes, Lugeon values and apparent Lugeon values, and plotting data into reports and CADD drawings.

2.5.4.5.1.2.2 Permeation Grout Testing Program

A Grout Test Program has been performed and the results have been finalized. Though grouting is not safety-related, mix design, material control, laboratory testing, grout placement, and field testing were conducted to meet NQA-1 quality requirements.

Mix designs were established for the various grout types, indicating the proportions of material constituents, as well as the target design parameters. All grout mixes were a combination of water, cement, flyash, bentonite, and superplasticizer. Mortar grout mixes or "low mobility sand grout mixes" were not used.

A Grout Test was implemented to specifications consistent with the design parameters set forth in this FSAR. The Grout Test Program consisted of nineteen grout holes arranged in a hexagonal pattern, including seven "Primary" grout holes of a higher viscosity grout, and twelve "Secondary" grout holes of a lower viscosity grout. These 19 holes were upstage and downstage grouted from a depth of 141 ft. bgs to a depth of 66 ft. bgs, as prescribed for the large-scale foundation grouting effort.

The purpose of the Grout Test Program was to validate the grout design and grouting techniques, to measure the change in the shear wave velocity and permeability of the grouted zone, and to determine the grout take in the Avon Park Formation.

The grout holes were installed using an automated real-time monitoring system for the water pressure testing and grouting, capable of computing a suite of engineering data allowing side-by-side evaluation of geology, grout mixes, Lugeon values and apparent Lugeon values, and plotting data into reports and CADD drawings.

Six initial and final verification core holes were drilled and water tested to verify pre- and post-test conditions. Prior to the commencement of and upon completion of the Grout Test Program, P-S suspension logging was performed to determine the effect of the grouting on the stiffness of the grouted mass. Because no appreciable change in shear wave velocity was observed post-grouting, the increased stiffness of the grouted zone will still be bounded by the randomization used in the site response analysis, as discussed in FSAR Subsection 2.5.2.5.1.

2.5.4.5.2 Excavation Extents

After the installation of the diaphragm walls and grouting operation described in FSAR Subsection 2.5.4.5.1, LNP 1 and LNP 2 will be vertically excavated to the approximate location of the Avon Park Formation at elevation -7.3 m (-24 ft.) NAVD88. The diaphragm walls serve as the excavation limits for the nuclear island.

Figures 2.5.4.5-201A and 2.5.4.5-201B show the planned nuclear island excavation limits at LNP 1 in plan view, and along southwest to northeast ("Plant South" to "Plant North") cross sections, respectively. Figures 2.5.4.5-202A and 2.5.4.5-202B indicate this same information for LNP 2.

Since the top of the Avon Park Formation is an erosional surface, its elevation is expected to be undulatory. As discussed in FSAR Subsection 2.5.4.5.3, elevation -7.3 m (-24 ft.) NAVD88 has been selected as a target elevation for nuclear island subgrade improvement.

Seismic Category II and nonsafety-related structures adjacent to the nuclear island will be supported on drilled shaft foundations. Considering the soil conditions at the site and the anticipated structural loads, shallow foundations will not provide adequate bearing capacity within permissible settlement and differential settlement requirements, and soil improvement techniques are not recommended due to the high water table and wetland conditions at the site. The conceptual design of the drilled shafts and installation is summarized in FSAR Subsection 3.8.5.9. Foundation design concepts under Seismic Category II and nonsafety-related structures adjacent to the nuclear island are shown on Figures 2.5.4.5-201A, 2.5.4.5-201B, 2.5.4.5-202A, 2.5.4.5-202B, 3.7-209, and 3.7-226.

2.5.4.5.3 Excavation Methods and Subgrade Improvement

As previously discussed in FSAR Subsection 2.5.1, the Suwannee and Ocala limestone formations are absent from the site, creating a geologic unconformity between the Avon Park Formation and the overlying undifferentiated Quaternary and Tertiary sediments. The Suwannee and the Ocala formations were eroded away, creating an erosion surface at the top of the older Avon Park Formation. This erosion surface is undulatory and the zone of the unconformity is of variable thickness. Careful reviews of the FSAR geotechnical and geophysical investigations were conducted, with consideration given to RQD, core recovery, SPT blow counts, shear-wave velocity, and overall condition of the core samples. A geologic and engineering interpretation was made that subsurface materials below elevation -7.3 m (-24 ft.) NAVD88 exhibit more desirable properties for foundation suitability than the materials above this elevation.

At both LNP 1 and LNP 2, rock at the nuclear island subgrade elevation -7.3 m (-24 ft.) NAVD88 will need to satisfy the following criteria:

- Rock will be moderately to highly cemented (naturally).
- Subgrade will not have solution features, loose rock, or open or soil-filled joints or fractures.

Foundation rock at elevation -7.3 m (-24 ft.) NAVD88 that does not satisfy these criteria will be removed and replaced or improved. A detailed excavation, subgrade improvement, and verification program will be developed prior to construction. The program will include the following general items:

- Specification of excavation methods. It is anticipated that excavation methods will include mass excavation of soils and highly weathered rock, and ripping of moderately weathered rock.

- Quality control and quality assurance programs.

- Methods for dewatering and protection of the subgrade from degradation during excavation and dewatering. Anticipated construction dewatering requirements are discussed in FSAR Subsection 2.5.4.6.2. It is not expected that the sound rock at the subgrade elevations will significantly degrade due to excavation, dewatering, or exposure to the elements during construction. Any degraded rock at subgrade elevation will be removed or improved prior to placement of dental concrete or the mudmat.

- Specification of methods for construction dewatering, disposal of water, and management of seepage and piping.

- Complete geologic mapping of the excavation, where exposed, will be undertaken prior to and during subgrade improvement activities. This mapping will occur in stages as the excavation is advanced. Due to the presence of the diaphragm wall and the inherent instability of the undifferentiated Quaternary sediments (if vertically excavated), the excavation will occur in 3 m (10 ft.) vertical increments at a 2 to 1 (horizontal to vertical) slope, inside the boundaries of the each diaphragm wall. The face of the slope created will be mapped, and then the slope will be removed, exposing the diaphragm wall. This process will be repeated in 3 m (10 ft.) increments down to the final excavation level. Complete geologic mapping of the excavation bottom will occur at the appropriate time.

- Excessively fractured or weathered rock will be over-excavated to the bottom of the weathered or fractured zone and filled with dental concrete.

- Soil-filled joints or fractures will be washed free of soil infilling to at least 1.5 m (5 ft.) below subgrade and filled with dental concrete.

- The inspection and mapping of the completed excavations will be performed by appropriately qualified and trained project inspection personnel. Soundings, test holes, and similar measures will be used to augment visual identification of areas needing repairs and to document acceptance of corrective measures, as appropriate.

Milestones for the excavation, subgrade improvement, and verification program are not identified at this time, but will be developed in conjunction with detailed design and construction planning. Additional description of foundation design is provided in FSAR Subsection 2.5.4.12.

2.5.4.5.4 Properties of Backfill Beneath and Adjacent to Nuclear Island

Based on a design grade elevation of 15.5 m (51 ft.) NAVD88, the elevation of each nuclear island basemat will be 3.4 m (11 ft.) NAVD88. A 15.2 cm (6 in.) mudmat will be located beneath each nuclear island basemat at elevation 3.4 m (11 ft.) NAVD88. Structural fill between the excavation bottom (elevation -7.3 m [-24 ft.] NAVD88) and the nuclear island mudmat (elevation 3.4 m [11 ft.] NAVD88) will consist of an RCC bridging mat, as shown on Figures 2.5.4.5-201B and 2.5.4.5 202B. A waterproofing membrane will be located between the RCC and the mudmat, meeting AP1000 DCD requirements of 0.55 static coefficient of friction between horizontal membrane and concrete. For buildings adjacent to the nuclear islands, the design grade will be raised to elevation 15.5 m (51 ft.) NAVD88 using engineered fill.

The following is the Design Description of the RCC. This RCC fill will serve two purposes: 1) replace the weakly cemented, undifferentiated Tertiary sediments that are present above elevation -7.3 m (-24 ft.) NAVD88, thereby, creating a uniform subsurface with increased bearing capacity; and 2) bridge conservatively postulated karst features.

The RCC bridging mat has been designed to bridge a 3-m (10-ft.) air-filled cavity located immediately beneath the RCC (elevation -7.3 m [-24 ft.] NAVD88) at any plan location for loading conditions identified in DCD Tier 1 Table 5.0-1 and Tier 2 Table 2-1. The 1-year specified compressive strength (f'_c) of the RCC is 2500 psi. The design of the RCC bridging mat has considered a nominal tensile strength of 250 psi.

A theoretical rock profile for the North and South Plant Units was developed using LNP site-specific rock properties and layering information. A SAP2000 Finite Element Model (FEM – linearly elastic) of the RCC, nuclear island basemat, and the subsurface rock was created using the design geometry, the rock profile beneath the RCC Bridging Mat, and the total loads applied by the nuclear island.

Also included in the FEM was the presence of theoretical cavities of different sizes and configurations. Three different cases, with cavities located at different depths, were considered:

- Case A: Cavities were located immediately below the grouted limestone, at elevation -99 ft. NAVD88 (75 ft. under the RCC).
- Case B: Cavities were located immediately below the RCC, at elevation -24 ft. NAVD88.
- Case C: Cavities were located at the top of rock layer NAV-3, which is the layer with lower Elastic Modulus for the North Reactor profile, below elevation -149 ft. NAVD88 (125 ft. under the RCC). This case was analyzed only in the North Reactor, where the lower Elastic Modulus layer is somewhat thicker than in the South Reactor profile.

Examples of the locations of these cavities are shown on Figure 2.5.4.5-204.

Sample stress plots and result tables were generated for the maximum stresses derived from the different cases of this analysis.

RCC bridging mat will be constructed using unreinforced RCC. Neither the AP1000 DCD nor ACI 349-01 addresses requirements for unreinforced (plain) concrete. ACI 349-01 specifies load factors and strength reduction factors for nuclear safety-related concrete structures. ACI 318-99 (Chapter 22) provides design methodology for unreinforced (plain) concrete. Thus, for the RCC bridging mat design, load factors, and strength reduction factors from ACI 349-01 and methodology from ACI 318-99 Chapter 22 are used for compressive and tensile capacity. For shear stress across lift joints, the strength is represented by a Mohr envelop relationship as described in USACE EM 1110-2-2006. A safety factor of 2.17 was then applied to ensure adequate performance. The 2.17 factor of safety incorporates both the load factor and the strength reduction factor for plain concrete.

The Pre-COL RCC testing performed and the Post-COL RCC Testing planned is described in FSAR Subsection 3.8.5.11. The RCC testing is to verify that the specified 2500 psi RCC compressive strength, ACI 318-99 (Chapter 22) specified tensile strength, and USACE EM 1110-2-2006 specified shear strengths across lift joints can be achieved.

The nuclear island vertical load considered in the analysis is 287,000 kips. The total vertical load of 287,000 kips corresponds to an average uniform load of 8.93 ksf, which exceeds the DCD Tier 1 requirements for bearing capacity. For the RCC bridging mat analysis, 70 percent of the total vertical load was considered dead load, and 30 percent was considered live load.

In the 3-D FEM, the shear forces were fully transmitted between the basemat and the RCC and between the RCC and the subsurface rock.

In the 3-D FEM, the subsurface material (limestone) that was included in the model below the RCC was sufficiently extended in both lateral direction and depth so that at the borders of the model, the stresses and deformations, due to the external loads applied to the NI basemat, are relatively small.

Any additional strength provided by grouting the upper 75 ft. of limestone was conservatively not included in this analysis. The rock mass properties (ungrouted) for that layer were used.

The LNP 2 profile presented lower values of rock mass elastic modulus; therefore, in most cases, the resulting tensile stresses were higher in LNP 2 than in LNP 1.

Controlled low strength material fill will be placed adjacent to the sidewalls of the nuclear islands to an elevation at least 1.5 m (5 ft.) below the bottom of the

adjacent buildings' foundation mat. Engineered fill will be placed from the top of the controlled low strength material fill to the bottom of the foundation mats of the adjacent Turbine Building, Annex Building, and the Radwaste Building. Figure 3.7-226 shows the approximate planned limits of controlled low strength material fill adjacent to nuclear island structures at LNP 1 and LNP 2.

Table 2.5.4.5-201 is a summary of the anticipated engineering properties for each backfill type. The characteristics and use of the materials described in Table 2.5.4.5-201 are as follows:

- RCC fill. This will consist of a roller compacted concrete bridging mat to be used to replace undifferentiated Tertiary sediments and to bridge conservatively postulated karst features.

- Controlled low strength material (CLSM) fill will be placed adjacent to the sidewalls of the nuclear islands to an elevation at least 1.5 m (5 ft.) below the bottom of the adjacent buildings' foundation mat as shown in Figure 3.7-226.

- Engineered fill will be used under the footprint of the TB, AB, and RB and to a lateral extent of ~30 ft. beyond the building footprint as discussed in FSAR Subsection 3.7.1.1.1 and shown in Figure 3.7-208. Engineered fill will also be placed from the top of the controlled low strength material fill to the bottom of the foundation mats of the adjacent Turbine Building, Annex Building, and the Radwaste Building as shown in Figure 3.7-226.

The engineering properties listed in Table 2.5.4.5-201 will be included in the construction specifications. Engineered fill material sources, once identified, will be mix-designed and tested to demonstrate that they are consistent with the properties in Table 2.5.4.5-201. The development of the CLSM fill specification and associated testing will occur prior to construction.

- Criteria for determination of nuclear island allowable bearing pressures, including analysis methods and selection of conservative rock strength parameters, are presented in FSAR Subsection 2.5.4.10.1. Selection of static and dynamic factors of safety is presented in FSAR Subsection 2.5.4.10.1.

- Criteria for determination of nuclear island settlement and subgrade rebound, including analysis methods and selection of conservative rock and soil parameters, are presented in FSAR Subsection 2.5.4.10.3. Tolerable settlement limits are presented in FSAR Subsection 2.5.4.10.3.3.

- Criteria for estimation of nuclear island sidewall lateral earth pressures are presented in FSAR Subsection 2.5.4.10.4.

For engineering analyses supporting the design and evaluation of safety-related structures, each software package used was validated and verified to operate properly on the computers used for the analyses in accordance with the Paul C. Rizzo Associates, Inc., Quality Assurance program. Specific software packages used for these analyses are described in the above-referenced design criteria subsections.

2.5.4.12 Techniques to Improve Subsurface Conditions

LNP COL 2.5-7 Major structures will derive support from the Avon Park Formation, at elevation -7.3 m (-24 ft.) NAVD88. Prior to excavation, grouting will be performed between this foundation elevation and elevation -30.2 m (-99 ft.) NAVD88 to create a relatively impervious zone of limestone to facilitate dewatering during construction.

Prior to the excavation of the nuclear island foundations, grout holes will be drilled from the existing ground surface to the proposed bottom of the targeted grout zone (elevation -32 m [-99 ft.] NAVD88). Grouting will be performed using a suite of mixes developed in the Grout Test Program. Primary grout holes will be spaced on a 4.8 m (16 ft.) hexagonal pattern, and split-spaced with secondary grout holes to achieve “no take” conditions. Provisions will be in place to perform additional split-spacing to tertiary grout holes, as dictated by the performance of the production grouting. This hole spacing was developed based on the results of the Grout Test Program conducted in early 2009. State-of-the-practice computerized monitoring of all grouting will take place, including the measurement of grout take in terms of pressure and volume.

Grouting will reduce the gross porosity and the gross permeability of the Avon Park Formation in this grouted zone. An additional benefit of this grouting is the long-term reduction of groundwater flow through the formation and the consequential reduction in the potential for renewed solution activity. This grouting program is not intended to strengthen the formation. However, the improved strength of the Avon Park Formation will add conservatism to the

design. Grouting is nonsafety-related; however, it will be performed under a quality program.

Upon completion of the grout program and dewatering effort, the nuclear island foundations will be excavated to the interpreted top of the Avon Park Formation at elevation -7.3 m (-24 ft.) NAVD88. Sound rock is present at this elevation, which is capable of supporting the structures with surface repairs and dental concrete as necessary to level this erosional surface. Criteria for acceptable subgrade conditions are presented in FSAR Subsection 2.5.4.5.3. Rock that does not satisfy the criteria will be removed and replaced with concrete or grout.

Subsequent to the excavation described in FSAR Subsection 2.5.4.5.3, a RCC Bridging Mat will be constructed at elevation -24 ft. The mat will be installed in 1-ft. lifts to elevation 11 ft. The extent that the RCC placement is shown on Figure 2.5.4.5-201A and Figure 2.5.4.5-201B for LNP 1, and Figure 2.5.4.5-202A and Figure 2.5.4.5-202B for LNP 2.

The RCC will be placed in lift thicknesses of approximately 1 ft. Bedding Mix will be used over each entire lift surface for the RCC bridging mat construction. The Pre-COL RCC testing performed and the Post-COL RCC Testing planned is described in FSAR Subsection 3.8.5.11.

The specified density of RCC is in the range 143 to 153 pcf. During the construction of the RCC Bridging Mat, field measurements of RCC density will be performed using a “single-probe nuclear densometer” for each 1-ft. lift during placement of the RCC.

Verification laboratory tests will be performed to confirm that the compressive strength of the RCC is satisfactory. The tests will be conducted using six-inch cylindrical test specimens molded during construction, in accordance with ASTM C 1435/C 1434M-05: “Standard Practice for Molding Roller-Compacted Concrete in Cylinder Molds Using a Vibrating Hammer”. Concrete to make the test specimens will be taken from six different locations for each 1-ft. lift of the RCC. Three samples will be taken at each of the six locations. The compressive strength tests will be conducted within 1 year of placement of the RCC. Compressive strength testing will be performed in accordance with ASTM C 39 “Test Method for Compressive Strength of Cylindrical Concrete Specimens.” All laboratory testing will conform to NQA-1 quality requirements. The strength level of RCC, adjusted for aging, will be considered satisfactory if either conditions 1 and 2 or conditions 1 and 3 are satisfied:

1) The average of compressive strength from three cylinders molded at a location equals or exceeds f’c.

2) No individual strength test (average of two cylinders) falls below f’c by more than 500 psi.

3) If individual strength tests (average of two cylinders), adjusted for aging, fall below f’c by more than 500 psi, a minimum of three cores drilled from the area in question shall be tested. The cores shall be drilled in accordance with ASTM C42: “Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.” RCC in areas represented by core tests shall be considered adequate if the average of compressive strength from three cores is equal to at least 85 percent of f’c and if no individual core compressive strength is less than 75 percent of f’c.

If these acceptance criteria are not met, an evaluation of the acceptability of the RCC for its intended function shall be performed before acceptance.

A detailed excavation, subgrade improvement, and verification program will be developed prior to and during construction. Subgrade improvement and verification methods summarized in FSAR Subsections 2.5.4.5.3 and 3.8.5.11, or equivalent, will be included in this program. The operational monitoring program for LNP 1 and LNP 2 is described in FSAR Subsection 2.4.12.4.

2.5.4.12.1 Impact of Dissolution Rate

As discussed in FSAR Subsection 2.5.4.1.2.1.1.1, the current dissolution rate of the Avon Park Formation is insignificant with regards to the foundation design. The operation of LNP’s production wells, after full installation of the AP1000 basemat, RCC Bridging Mat, and grouted zone, was shown to have little significant impact on the groundwater regime of the site. Compared to the natural regime at the site, the LNP construction was shown to impact the hydrology approximately the same as the seasonal fluctuations. Given this and the very low expected dissolution rates described in FSAR Subsection 2.5.4.1.2.1.1.1, the potential for increased dissolution as a result of construction is also insignificant.

2.5.5 STABILITY OF SLOPES

LNP COL 2.5-14 The nominal plant grade floor elevation at the LNP site will be at 15.5 m (51 ft.) NAVD88, with minor variations to allow drainage for an area of about 370 m by 390 m (1210 ft. by 1280 ft.) around the nuclear island. No permanent slopes will be present at the site that could adversely affect safety-related structures.

LNP COL 2.5-15 The AP1000 does not utilize safety-related dams or embankments, and there are no existing upstream or downstream dams that could affect the LNP site safety-related facilities.

The potential for unacceptable deformation of the AP1000 basemat, related to karst features immediately beneath the plant structures is eliminated with these measures in place.

2.5.4.1.2.2 Uplift or Collapse

The LNP site is located on the west coast of the Florida platform. As described in FSAR Subsection 2.5.1.1.2.3, the Florida platform represents long-term carbonate sedimentation on a passive margin, and late Quaternary deposits have not experienced significant uplift, subsidence, or tectonic deformation. (Reference 2.5.1-236)

2.5.4.1.3 Zones of Alteration, Irregular Weathering, or Structural Weakness

The bedrock, which underlies the undifferentiated Quaternary sediments, is the middle Eocene-aged Avon Park Formation. The upper portion of this formation, which consists of calcareous silts (units S2 and S3, also referred to as undifferentiated Tertiary sediment) appears to have been altered by weathering and greater degrees of dissolutioning (FSAR Subsection 2.5.1.2). This zone occurs near elevation -7.3 m (-24 ft.) NAVD88, although it is highly undulatory by nature. A review of the boring logs indicates that the base of the zone varies by up to approximately 2.1 m (7 ft.) below elevation -7.3 m (-24 ft.) NAVD88 within the extents of the nuclear islands. These undifferentiated sediments will be excavated, and the excavation surface will be cleaned and prepared as described in FSAR Subsection 2.5.4.5.3.

Zones of structural weaknesses, such as extensive fractured or faulted zones, are not present; however, vertical joints, sometimes connected by horizontal bedding planes, are present and may act as zones of weakness. These discontinuities, likely a factor in the localization and development of dissolution activity at the site, were considered in the rock mass properties used in design.

2.5.4.1.4 Unrelieved Stresses in Bedrock

There is no evidence of unrelieved stresses in bedrock, as noted in FSAR Subsection 2.5.1.2.6.

2.5.4.1.5 Rocks or Soils that may Become Unstable

The potential hazard from rocks or soils that may become unstable was determined to be low. While evidence of historic solution activity is present, FSAR Subsections 2.5.4.1.2.1, 2.5.4.5, and 2.5.4.10 present background information, construction techniques, and engineering analyses, which indicate the mitigation of any significant future solution activity immediately beneath the nuclear island. The potential for liquefaction is presented in FSAR Subsection 2.5.4.8.

2.5.4.6 Groundwater Conditions

LNP COL 2.5-6
LNP COL 2.5-8

Groundwater conditions for the LNP site were established by periodic measurements of groundwater levels since well installation in 2007, as summarized in FSAR Subsection 2.4.12. Data from these measurements provide a basis for engineering design and for preliminary construction dewatering plans.

2.5.4.6.1 Groundwater Elevations

Groundwater elevations at site monitoring wells are presented in FSAR Subsection 2.4.12. Water-table data collected in 2007 indicates that the water table ranges in depth at LNP 1 and LNP 2 areas from less than 0.3 m (1 ft.) bgs during rainy periods to approximately 1.5 m (5 ft.) bgs during drier periods.

As described in FSAR Subsection 2.4.12.5, post-construction groundwater elevations are not anticipated to exceed elevation +14.6 m (+48 ft.) NAVD88. The nuclear islands will be founded on RCC over rock; groundwater conditions are not expected to adversely affect foundation performance.

2.5.4.6.2 Construction Dewatering

Dewatering will be required to maintain groundwater levels beneath the nuclear island to an elevation of -7.3 m (-24.0 ft.) NAVD88 or lower during excavation and construction of the nuclear islands. Expected construction dewatering flow rates and anticipated dewatering methods are summarized in this subsection.

Due to the size of the excavation, as well as the expected quantity of groundwater that could potentially be encountered, a diaphragm wall will be constructed around each entire nuclear island to minimize lateral groundwater inflow into the excavation. In addition, the Avon Park Formation will be drilled and pressure grouted to elevation -30.2 m (-99 ft.) NAVD88 before excavation begins to minimize seepage from the rock upward into the excavation, and to resist possible uplift pressure. The diaphragm wall will be keyed approximately 9.1 m (30 ft.) into the grouted bedrock. Thus, two different engineered barriers will form a “bathtub” with the diaphragm wall being the sides and the grouted Avon Park Formation being the bottom of the “bathtub.” With this design, one has to dewater the excavation area with relatively shallow wells and sumps within the area.

Expected construction dewatering pumping rates were calculated using the Visual MODFLOW software package, which includes the U.S. Geological Survey’s three-dimensional finite-difference modeling code, MODFLOW 2000. A three-dimensional model was constructed that has six horizontal layers, each of uniform thickness. The top layer is 20.4 m (67 ft.) thick and represents the undifferentiated Quaternary and Tertiary sediments (fine sand and silty sand). The second and third layers represent the uppermost 22.9 m (75 ft.) of Avon Park Formation, which is grouted beneath the nuclear island. The lowermost four layers of the model represent the middle portion of the Avon Park Formation, which is ungrouted. Together, Model Layers 2 through 6 represent the permeable

portion of the Avon Park Formation, which in this area is also referred to as the Upper Floridan aquifer. The base of the model was set at -137 m (-450 ft.) NAVD88.

The diaphragm wall penetrates through the undifferentiated surficial sand (Model Layer 1) and the uppermost 9.1 m (30 ft.) of Avon Park Formation (Layer 2 in the model). Figure 2.5.4.6-201 shows a cross section of the model and displays how the diaphragm wall and the grouted Avon Park Formation are represented in the model.

The entire model area (one unit) is 1174 m by 1174 m (3850 ft. by 3850 ft.) in area. The excavation area and the diaphragm wall occupy an area of 82.3 m (270 ft.) long by 51.8 m (170 ft.) wide for each unit and are located in the center of the model area. Only one unit is modeled because the units will be dewatered at different times.

Table 2.5.4.6-201 lists the hydraulic characteristics of each geological material represented in the model. The hydraulic conductivity values used in the model are presented in FSAR Subsection 2.4.12. Average values were obtained from hydraulic conductivity field tests (FSAR Subsection 2.4.12).

The model accounts for the results of a sensitivity analysis conducted to determine the susceptibility for increased flow to the shallow interior wells based on postulated leakage “windows” that develop in the diaphragm wall. While state-of-the-art diaphragm wall construction has reduced vulnerability to such defects (such as panels rotating, assuming a degree of curvature, or otherwise not aligning adequately), sump pumps located at the bottom of the excavation will be pumped to address any potential “window leakage,” as well as rainfall and surface runoff during the excavation process.

The gross permeability of the diaphragm wall is taken as 10^{-6} cm/sec (0.002835 ft/day). Potential leakage through “windows,” as mentioned above, may necessitate greater than expected pumping rates in order to maintain dry working conditions within the excavation. The permeability of the grouted Avon Park Formation has been conservatively considered to be 10^{-4} cm/sec (0.2835 ft/day), but this parameter has been varied to account for the possible variation in the effectiveness of the grouting operation.

The hydraulic conductivity of the ungrouted Avon Park Formation at the LNP site ranges from 8.47×10^{-4} to 1.92×10^{-2} cm/sec (2.4 to 54.4 ft/day), and averages approximately 4.9×10^{-3} cm/sec (13.9 ft/day).

The total flow that must be accommodated with sumps and shallow wells is conservatively determined to be in the range of 1136 to 1893 lpm (300 to 500 gpm) at steady-state conditions during construction, based on the site hydraulic conductivity characteristics summarized in Table 2.5.4.6-201 and the hydrogeological conditions at the site, as described in FSAR Subsection 2.4.12.

The groundwater pumping rate during excavation can be managed by six submersible pumps (each with 379 lpm [100 gpm] capacity) installed in wells located around the inside perimeter of the diaphragm wall and the grouted zone with pumps placed in sumps within the excavation.

When the excavation has reached its target depth, the exposed rock and the diaphragm wall will be inspected and evaluated for leakage. In the event that significant leakage is observed (e.g., greater than 379 lpm [100 gpm]), a second round of drilling and pressure grouting at specific locations will be implemented to seal areas where groundwater is seeping through the engineered barriers.

During construction, a groundwater monitoring program will be implemented to monitor the head differential between the inside and the outside of the diaphragm wall, as well as the uplift pressure on the bottom of the excavation, as described in FSAR Subsection 2.5.4.10.3.5.

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by Eocene limestone, and covered by thin Pleistocene sands deposited by the regressing Gulf of Mexico, as described in FSAR Subsection 2.5.1.2.1.1.

The boring logs presented in Appendix 2BB indicate that the LNP site subsurface is consistent with the Gulf Coastal Lowlands geomorphic province. The results of petrographic examinations of rock core samples, as described in FSAR Subsection 2.5.4.2.3.2.2, are also consistent with rock of this geomorphic province. Of the 20 petrographic samples that were examined, 18 were identified as completely dolomitized limestone.

2.5.4.1.1.2 Dip of Rock Strata

As discussed in FSAR Subsection 2.5.4.4.2.2, the weighted global mean dip of bedding planes is approximately horizontal at LNP 1 and LNP 2. The data from this analysis are presented in Table 2.5.4.4-202.

2.5.4.1.2 Subsidence, Dissolution Activity, Uplift, or Collapse

As described in FSAR Subsection 2.5.1.2.5, there is no record that human activities, such as mining, have been performed in soil or rock near the vicinity of the LNP site, and hence there is no risk associated with mine subsidence or collapse.

The potential for subsidence or collapse pertaining to future solution activity of the LNP subsurface is described in this section, as well as the potential for uplift.

2.5.4.1.2.1 Dissolution Activity

Movement of water through a carbonate rock formation is a catalyst for dissolution activity. The ability for water to move within a rock mass requires either jointing, fractures, or porous characteristics to facilitate movement. As these features become hydraulically interconnected by chemical dissolution of the rock, they enhance the movement of groundwater. At greater depths within the aquifer, the regional aquifer gradients drive the groundwater movement in a predominantly horizontal flow path towards the aquifer’s ultimate discharge into the Gulf of Mexico.

Based on results of field investigations, the Floridan aquifer system in the Avon Park Formation at the LNP site consists of interbedded carbonate rock units such as fossiliferous limestone, dolomitized limestone and dolomite. The permeability of these interbedded units is generally high but does vary due to differences in rock texture (primary porosity), secondary fossiliferous porosity, amount of fracturing and degree of dolomitization of the limestone, among other factors. Dolomitization may either increase or decrease the porosity of the rock, but the recrystallization that occurs during dolomitization can result in increased density of the rock. Once limestone has been converted to dolomite, there is less potential for future dissolution of the rock by groundwater.

Specific Avon Park Formation aquifer zones at the LNP site were evaluated using downhole geophysical logging, exploratory borehole rock coring logs, acoustic televiewer logs, and seismic geophysical testing results. Summaries of these testing results are presented in FSAR Subsections 2.5.4.2.1.1, 2.5.4.4.2.1, 2.5.4.4.2.2, and 2.5.4.4.2.3. Karst features at the LNP site, in a manner consistent with Florida geology, typically exhibit a "plus-sign" morphology, whereby chemical dissolution activity in the limestone occurs along both vertical and horizontal planes (vertical fractures and horizontal bedding planes). As described in detail in FSAR Subsection 2.5.4.4.2.5, the gamma-gamma geophysical logs identified randomly distributed lower-density zones that have no spatial significance (i.e., two such low-density zones do not occur at the same depth in adjacent borings). The low-density zones generally have material present and were not voids. The thickness of these possible karst features is typically limited to less than 1.5 m (5 ft.). These features are likely associated with vertical/near-vertical fractures.

The results of the neutron-neutron (porosity) logging, as described in FSAR Subsection 2.5.4.4.2.6, consistently identified the presence of a lower-porosity zone relative to surrounding rock at depths of approximately 42.7 m to 57.9 m (140 ft. to 190 ft.) bgs at LNP 2, and to a lesser extent, also at LNP 1. The downhole seismic studies of formation shear-wave velocities also indicate a slightly higher velocity in the 42.7-m to 57.9-m (140- to 190-ft.) depth zone. Rock coring results in this interval also indicate generally better recoveries and higher rock quality designations (RQD) than zones immediately above or below this depth interval.

These findings indicate that the carbonate rocks of the Avon Park Formation in the 42.7-m to 57.9-m (140-ft. to 190-ft.) depth interval are less susceptible to rapidly developing karst activity associated with vertical infiltration of surface water than the rock units above this interval. This zone within the aquifer is also more dolomitized, displays relatively lower porosity characteristics based on geophysical logging, is more competent structurally, and dissolution activity in this zone will be primarily limited to the gradual processes caused by groundwater movement throughout the aquifer.

There is some indication of dissolution activity within horizons at deeper depths at the LNP site, based on petrographic thin section analysis of rock core samples. The deepest paleosinks observed at or near the LNP site lie beyond the footprints of the nuclear islands and extend to depths of approximately 73 m (240 ft.) bgs (approximately -61 m [-200 ft.] NAVD88), thereby indicating the possible base of the epikarstic horizon at the LNP site. This 73-m (240-ft.) depth coincides with a low seismic velocity zone (site response sublayer Av3b, Figure 2.5.1-250) observed in the shear (V_s) and compression (V_p) wave velocity data obtained at the LNP site. Some of the decreased density and lower velocities observed in this horizon indicate dissolution, but it is noted that the shear-wave velocities in this zone are still in the range of 2100 to 3500 ft/sec, and the material properties may reflect instead the original depositional properties of the rock (e.g., the presence of small interbeds of silt within this unit suggest a different depositional environment than horizons above or below this unit). This is

consistent with the findings of the offset boring program that the soft beds within the Avon Park Formation are zones of severely weathered to degraded dolomite, and not cohesive soils. Fewer karst features were encountered in this zone than in the horizons above a depth of approximately 45.7 m (150 ft.) bgs.

Weaker zones of weathered or organic material below the elevation of -61 m (-200 ft.) NAVD88 also were observed as described in FSAR Subsection 2.5.4.4.2.5, but given the depth (approximately 132 m [433 ft.] bgs) and size (about 18 cm [7 in.]) of these features, they are judged to be insignificant to the design of the facilities.

The following subsections describes the nature, frequency, thickness, and lateral extent of the postulated features of dissolution at the LNP site.

2.5.4.1.2.1.1 Nature of Features

As described in FSAR Subsection 2.5.1, Vernon describes a regional fracture set trending NW-SE and NE-SW. (Reference 2.5.1-261) Others have observed a second ENE-WSW and WNW-ESE fracture set, as described in FSAR Subsection 2.5.3.2.1.

A subset of these regional fractures has been identified during subsequent field investigation, with primary and orthogonal fracture spacing on the order of 5.8 to 7.2 m (19 to 23.5 ft.). This fracture set has been observed in local outcrops near the LNP site during field reconnaissance at the Gulf Hammock quarry (located approximately 19 km [11.8 mi.] NNW from the LNP site), and along the banks of the Waccasassa River (located approximately 25 km [15.7 mi.] NNW from the LNP site). In addition, exposed orthogonal vertical fractures of approximately N-S and E-W strikes have been observed as the local dominating joints at the Gulf Hammock quarry and as the less prominent subset along the banks of the Waccasassa River. High-angled joints were also observed during the site investigation. The vertical joints were observed to be on the order of 0.6 to 1 m (2 to 3 ft.) wide at the surface and diminishing in width with depth. The linear orientations of the land features in the area appear to be controlled by the two above-mentioned orthogonal joint sets. For example, the Waccasassa River flows in a north 6 degrees west orientation where the aforementioned joints were observed, and the Withlacoochee River flows west-northwest. Sections of both the Waccasassa and Withlacoochee Rivers appear to be controlled by aforementioned rock joints, as the bends are abrupt, and the sections are linear and distinct.

The Upper Floridan aquifer is a layered aquifer system, which produces water along zones near lithological contacts. These lithological contacts are where horizontal zones of weakness tend to occur. The variable Avon Park Formation contains layers or zones that are pervasively dolomitized in places and undolomitized in others. The types of secondary porosity (vugs and cavities, or fractures) differ depending on the degree of dolomitization of the rock. Limestone is more ductile and the apparent secondary porosity tends to be associated with vugs and cavities, whereas dolomite is harder and more brittle, and the

secondary porosity tends to be associated with fractures. As described in FSAR Subsection 2.5.4.2.3.2.2, 18 out of 20 samples of rock that were petrographically analyzed have been completely dolomitized.

As discussed in FSAR Subsection 2.5.4.1.2.1, the carbonate rocks of the Avon Park Formation are generally less susceptible to solution activity compared to the Ocala Formation that underlies much of Florida, including the Crystal River Plant (Reference 2.5.1-322). Furthermore, the Avon Park, at the LNP, in the 42.7-m to 57.9-m (140-ft. to 190-ft.) depth interval is less susceptible to karst activity associated with infiltration of surface water than the rock units above this interval. This depth interval within the aquifer is more dolomitized and displays relatively lower porosity characteristics based on geophysical logging. The dissolution process is described in FSAR Section 2.5.4.1.2.1.1.1.

2.5.4.1.2.1.1.1 Dissolution Process

The Crystal River 3 FSAR indicates that the Ocala Limestone present at the Crystal River site is dissolving at a rate of 1 x 10^{-4} percent per year, or 6 x 10^{-3} percent over 60 years. Due to high levels of dolomitization with recrystallization and the less soluble nature of dolomite than limestone, the Avon Park Formation is less susceptible to dissolution activity and consequential development of karst features than the Ocala Limestone. Given the insignificant rate of annual dissolution activity of the Ocala Limestone at the Crystal River Plant and recognizing that the LNP is founded on the Avon Park, the rate of dissolution activity at LNP is less than 1 x 10^{-4} percent per year.

2.5.4.1.2.1.2 Frequency and Thickness of Features

A review of the subsurface investigations data was conducted to evaluate the potential for karst features development within the limestone bedrock strata. The features were evaluated based on field observations during the rock coring, such as rod drop and circulation loss, as well as the recovered core and RQD data.

The results of this evaluation are presented in Tables 2.5.4.2-205A and 2.5.4.2-205B, "Summary of Karst Features Encountered in Boreholes." Depth and thickness of each feature are listed and summarized at each plant location and arranged by boring number.

The information is presented graphically on Figure 2.5.4.1-201A and Figure 2.5.4.1-201B, showing histograms of the thicknesses of the observed features. The thickness of an individual feature is typically less than 1.5 m (5 ft.).

It is noted that the histograms presented in Figure 2.5.4.1-201A and Figure 2.5.4.1-201B are based on the site characterization boreholes, which were largely concentrated in the upper 180 ft.The histogram is based on all of the available data and reflects the fact that there are more data available for the higher portion of the geologic profile simply because, as with all sites, there are more shallow borings and samples than deep borings and samples. Of course, there is a need for more data at shallow depths because the stresses induced by

foundations have to be accommodated by the shallower formations, whereas at deeper depths, the induced stresses diminish eventually to nil with depths on the order of 1.5 to 2 times the dimensions of the foundation being supported; hence less data are required at deeper depths. FSAR Subsection 2.5.4.10 demonstrates that potential features located below 180 ft. are of considerably less significance given the depth and the robustness of the foundation design, specifically the 35-ft. thick RCC Bridging Mat and the 6-ft. thick AP1000 basemat. The impacts of the subsurface below the upper 180 ft. are discussed in regard to bearing capacity, settlement, and other geotechnical parameters in FSAR Subsection 2.5.4.10.

2.5.4.1.2.1.3 Lateral Extent of Features

As described in FSAR Subsection 2.5.4.4.2.5, the gamma-gamma logs indicate randomly distributed low-density zones that have no spatial significance (i.e., two such low-density zones do not occur at the same depth in adjacent borings). The low-density zones generally have material present and are not voids. The thickness of these low-density zones is typically limited to less than 1.5 m (5 ft.).

The Avon Park Formation typically exhibits higher degrees of dolomitization than the late Eocene Ocala Limestone, and consequentially, less susceptibility to dissolution activity (Reference 2.5.1-322). Eighteen out of twenty (20) samples of rock that were petrographically analyzed have been completely dolomitized. This is significant because the more dolomitized Avon Park Formation layers have a higher percentage of recrystallized magnesium carbonate, and is therefore less susceptible to the types of karst activity known to occur within the pure calcium carbonate limestone zones typically present within the Ocala Limestone.

Vernon (Reference 2.5.3-203) describes a regional fracture set trending NW-SE and NE-SW. In March 2008, a subset of these regional fractures was identified during field investigations. This fracture set was observed in local outcrops near the LNP Site during field reconnaissance at the Gulf Hammock Quarry and along the banks of the Waccasassa River.

At the Gulf Hammock Quarry, along an Avon Park Formation outcrop striking due North, primary and orthogonal vertical fractures were observed. Fractures were evident at 30-ft. spacing along this outcrop, and had iron staining consistent with water infiltration along the fracture.

Along a portion of the Waccasassa River where the Avon Park Formation outcrops, striking at North 6 degrees West, primary and orthogonal vertical fractures were evident at 35-ft. spacing.

Given the strikes of these Avon Park Formation outcrops, and given the observed vertical fractures and spacing, a subset to the regional fracture set was postulated. This local fracture set, with primary fractures consistent with the North 39 degrees West strike associated with Vernon's regional fracture set, features a primary fracture spacing of approximately 19 ft. and an orthogonal fracture spacing of approximately 23.5 ft. The Avon Park Formation outcrop

strikes and the postulated fracture pattern associated with the local observed outcrops are shown on Figure 2.5.4.1-202.

In order to quantify the vertical dimension (thickness) of postulated karst features associated with these fracture sets, field observation data, gathered during rock coring, were evaluated. The results of this evaluation are presented in Table 2.5.4.2-205A and Table 2.5.4.2-205B and presented graphically on Figure 2.5.4.1-201A and Figure 2.5.4.1-201B. The thicknesses of these features are typically limited to less than 1.5 m (5 ft.).

Three methods were used to estimate the lateral dimension of the karst features on the LNP Site: field observations, geophysical testing, and excess grout takes from the subsurface investigation. Based on this analysis, the average width-to-height ratio of features associated with vertical fractures is 1H:5V, limiting the lateral extent of these features to approximately 20 percent of the vertical extent, as supported by geophysical testing and field observations. Dr. Anthony Randazzo, a subject matter expert, is supportive of the approach that the horizontal dimension is a fraction of the vertical dimension of the feature.

The largest single potential karst feature identified in Table 2.5.4.2-205A and Table 2.5.4.2-205B (19.5 ft.) would correspond to a vertical feature that is 20 percent of 19.5 ft., or 3.9 ft. wide.

Given the conservative estimations made in determining the lateral extent of the postulated karst features at the LNP Site, the RCC Bridging Mat was designed to span a 10-ft. diameter void beneath the Bridging Mat (elevation -24 ft. NAVD) at any plan location, at any depth. Additionally, following the offset boring program, it was determined that the Avon Park Formation contains fewer karst features than were conservatively postulated based on the original site characterization, as evidenced by the high recovery during the offset boring program. The soft beds within the Avon Park Formation have been identified and described as zones of severely weathered to degraded dolomite, and not the cohesive soils that were postulated following site characterization. Thus the design karst feature (10-ft. diameter void) is regarded as sufficiently conservative.

2.5.4.1.2.1.4 Mitigation of Potential Surface Deformation Related to Karst Features

Beneath the LNP 1 and LNP 2 nuclear islands, the uppermost approximately 20.4 m (67 ft.) of undifferentiated sediments and rock (to elevation -7.3 m [-24 ft.] NAVD88) will be excavated and backfilled with roller compacted concrete prior to construction, as described in FSAR Subsection 2.5.4.5.

As described in FSAR Subsection 2.5.4.5, the roller compacted bridging mat has been designed to span a 3-m (10-ft.) diameter void immediately beneath the bridging mat at any location within the footprint of the nuclear island, as well as a larger-diameter void immediately beneath the grouted zone.

The potential for unacceptable deformation of the AP1000 basemat, related to karst features immediately beneath the plant structures is eliminated with these measures in place.

2.5.4.1.2.2 Uplift or Collapse

The LNP site is located on the west coast of the Florida platform. As described in FSAR Subsection 2.5.1.1.2.3, the Florida platform represents long-term carbonate sedimentation on a passive margin, and late Quaternary deposits have not experienced significant uplift, subsidence, or tectonic deformation. (Reference 2.5.1-236)

2.5.4.1.3 Zones of Alteration, Irregular Weathering, or Structural Weakness

The bedrock, which underlies the undifferentiated Quaternary sediments, is the middle Eocene-aged Avon Park Formation. The upper portion of this formation, which consists of calcareous silts (units S2 and S3, also referred to as undifferentiated Tertiary sediment) appears to have been altered by weathering and greater degrees of dissolutioning (FSAR Subsection 2.5.1.2). This zone occurs near elevation -7.3 m (-24 ft.) NAVD88, although it is highly undulatory by nature. A review of the boring logs indicates that the base of the zone varies by up to approximately 2.1 m (7 ft.) below elevation -7.3 m (-24 ft.) NAVD88 within the extents of the nuclear islands. These undifferentiated sediments will be excavated, and the excavation surface will be cleaned and prepared as described in FSAR Subsection 2.5.4.5.3.

Zones of structural weaknesses, such as extensive fractured or faulted zones, are not present; however, vertical joints, sometimes connected by horizontal bedding planes, are present and may act as zones of weakness. These discontinuities, likely a factor in the localization and development of dissolution activity at the site, were considered in the rock mass properties used in design.

2.5.4.1.4 Unrelieved Stresses in Bedrock

There is no evidence of unrelieved stresses in bedrock, as noted in FSAR Subsection 2.5.1.2.6.

2.5.4.1.5 Rocks or Soils that may Become Unstable

The potential hazard from rocks or soils that may become unstable was determined to be low. While evidence of historic solution activity is present, FSAR Subsections 2.5.4.1.2.1, 2.5.4.5, and 2.5.4.10 present background information, construction techniques, and engineering analyses, which indicate the mitigation of any significant future solution activity immediately beneath the nuclear island. The potential for liquefaction is presented in FSAR Subsection 2.5.4.8.

Subsection 2.5.3). The postulated faults identified by Vernon (Reference 2.5.1-262) in the site vicinity, however, are not apparent in regional scale Landsat imagery or in more detailed aerial photograph mosaics (1949 black and white, 1:20,000 scale) that cover the site vicinity and area, respectively (see discussion in FSAR Subsection 2.5.3.2.1.1).

The postulated faults identified by Vernon (Reference 2.5.1-262) likely do not exist, and there is no evidence to suggest that they have been active in the Quaternary. Therefore, none of these postulated faults are considered to be capable tectonic sources, as defined in Regulatory Guide 1.208, Appendix C (see discussion in FSAR Subsection 2.5.3.6).

2.5.1.2.5 Geology of the Site Location

This subsection presents a more detailed discussion of the geologic conditions at the LNP site based upon field reconnaissance and subsurface exploration.

2.5.1.2.5.1 Site Location Geomorphology

The LNP site is situated in an area of pine plantation and cypress domes with wetlands (Figure 2.5.1-247). The original drainage and topography of the site have been modified by logging and silviculture activities over approximately the past 30 – 40 years. Aside from the logging operations and hunting trails, the site is undeveloped.

The general morphology of the LNP site location (1-km [0.6-mi.] radius) is illustrated by the detailed geomorphologic map derived from light detection and ranging (LIDAR) data as shown on Figure 2.5.1-248. The ground surface, which represents a broad, relatively flat marine terrace mantled by thin terrace cover sediments, increases from elevations of approximately 12 m (40 ft.) NAVD88 west of LNP 1 and LNP 2 to 14.6 m (48 ft.) NAVD88 at the eastern margin of the site location. The elevation of the ground surface at LNP 1 and LNP 2 is approximately 12.8± 1 m (42 ± 2 ft.) NAVD88.

The surface morphology is characterized by dolines (shallow depressions above sinks or paleosinks) varying in size from relatively small, (less than 50 m [164 ft.] in diameter) well-defined circular depressions to large (600 m [2000 ft.] wide) irregular, broad, shallow depressions that are more widespread in the western half of the site location. Many of the circular depressions, which are generally less than 1 to 2 m (2 to 6 ft.) deep, are coincident with cypress domes that are visible in both 1949 black and white (pre-extensive logging) and 2007 aerial photographs. The rectilinear pattern and linear margins of higher areas observed in the topography are consistent with regional joint trends. (See lineament analysis described in FSAR Subsection 2.5.3.2.1.1.)

The morphology is very similar to that of the present coastline in the northern part of Citrus County, which consists of rock-cored marsh islands, broad embayments, and joint-controlled tidal creeks that locally connect a series of circular sinkholes (Figure 2.5.1-236). This supports the conclusions of previous

researchers that the site area is underlain by older, karstified marine terrace surfaces mantled by a thin veneer of Quaternary sediment (i.e., References 2.5.1-308, 2.5.1-213, 2.5.1-201, and 2.5.1-235). The rectilinear pattern is more apparent in the terrain above an elevation of about 12.8 m (42 ft.) NAVD88. One possible explanation of the variable surface morphology is that the terrain to the west of the LNP units, which may be underlain by a younger (lower elevation) marine terrace, may have experienced a different erosional history. Alternatively, the variation in geomorphic expression across the site location may reflect shallower groundwater and a generally eastward increase in the thickness of Quaternary cover sand that was deposited against the Brooksville Ridge to the east of the site. The lack of well data for the undeveloped areas beyond the LNP site precludes identification of paleoshorelines and detailed mapping of the thickness of Quaternary cover that could be used to differentiate between these two hypotheses.

2.5.1.2.5.2 Site Location Stratigraphy

Stratigraphy of the LNP site location (1 km [0.5 mi.] radius) is known to a depth below ground surface of approximately -1387 m (-4551 ft.) from SWFWMD records of a deep oil and gas exploration well drilled as part of oil and gas exploration by Humble Oil in 1949. The well, known as Robinson No. 1, is located approximately 500 m (1640 ft.) north of the LNP north reactor (LNP 2) site (Figure 2.5.1-247). Site stratigraphy to a depth of 150 m (500 ft.) is known from geotechnical borings that were drilled as part of the COLA study. Details of the geotechnical boring program are provided in FSAR Subsection 2.5.4.

Well logs from Robinson No. 1 well indicate that Paleozoic age basement rock below the carbonate sequence at the LNP site is at a depth of approximately -1315 m (-4317 ft.), and that the total thickness of the Floridan aquifer carbonates is at least 610 m (2000 ft.) thick. The general stratigraphic sequence encountered in the Robinson No. 1 well (Table 2.5.1-203) consists of 9 m (30 ft.) of Quaternary sediments over approximately 870 m (2860 ft.) of interbedded dolostone and limestone, overlying 475 m (1560 ft.) of chalky limestone, shale and sand, which in turn lies above Paleozoic rock which has been intruded by a Mesozoic dike. The Paleozoic rock (quartzitic sandstone) was encountered at -1315 m (-4317 ft.) depth, and extended to the total drilled depth of -1387 m (-4551 ft.). The depths below ground surface from the Robinson Well No.1 log have been corrected in the FSAR for elevation of the drilling floor indicated on the driller’s log, and so do not exactly match the well depths reported in Vernon (1951), which were published uncorrected.

The generalized hydrostratigraphic column of Floridan aquifer system carbonate depositional sequence in west central Florida is illustrated in Figure 2.5.1-249. As shown in the figure, the Upper Floridan aquifer in the LNP site vicinity typically contains fresh potable water, and is separated physically and hydraulically from the underlying Lower Floridan aquifer by sequences of lower permeability evaporite rock units known as the Middle Confining Unit (MCU), which act as an aquitard. (Reference 2.5.1-323)

The results of the geotechnical boring program at the LNP site showed that the first carbonate rock units encountered below the surficial aquifer deposits (Unit S1) are limestone deposits (calcareous silts of Units S2 and S3; also referred to as undifferentiated Tertiary deposits) that are interpreted to be part of the middle Eocene age Avon Park Formation. Results of the subsurface investigation also indicate that the Miocene age Hawthorn Group is not present, and the Suwannee and Ocala Formations are also absent (Reference 2.5.1-324). This represents a geologic unconformity whereby all Miocene, Oligocene, and Late-Eocene age geologic deposits were eroded from the LNP site location. The contact between the S1 and S2 units is a time-stratigraphic unconformity (an erosional surface hereafter referred to as the “Quaternary/Tertiary (Q/T) unconformity”) that represents a gap in the depositional record of approximately 45 million years between the Quaternary surficial deposits and the underlying Avon Park Formation in this location (Figure 2.5.1-214).

Regional hydrostratigraphic studies show that the Avon Park Formation is underlain by the Oldsmar and Cedar Keys formations of Early Eocene and late Paleocene age, respectively. The Oldsmar and Cedar Keys formations are each expected to be at least 152 m (500 ft.) thick in the vicinity of the LNP site. The Avon Park Formation comprises the upper unit and the Oldsmar Formation comprises the lower unit of the Floridan aquifer system in this area; the Cedar Keys Formation acts as the lower confining unit for the Floridan aquifer system. To the maximum investigated depth of 152 m (500 ft.), neither the MCU nor the Lower Floridan aquifer units were encountered.

2.5.1.2.5.2.1 Site Stratigraphic Unit Descriptions

A generalized diagram showing the stratigraphic units encountered at the LNP site is shown on Figure 2.5.1-250. The generalized units shown correlate to layers used in the site response analysis, which were defined based on variation in shear-wave velocity, reviews of the lithology from the boring logs, and reviews of additional downhole geophysical measurements performed by Technos, Inc. (Reference 2.5.1-324) Correlation of the site response layers to stratigraphic layers defined primarily on engineering properties from the LNP 1 and LNP 2 boring program results (FSAR Subsection 2.5.4.2) are provided in the legend for Figure 2.5.1-250.

Surficial geologic deposits at the site consist of undifferentiated Quaternary age fluvial and terrace sediments, primarily silty fine quartz sands. The sands overlie the Avon Park Formation, a shallow marine carbonate rock unit of middle Eocene age.

The Quaternary deposits (designated unit S1) encountered in the LNP site borings generally consist of gray silty quartz sands. The subrounded to rounded sand grains and sorting indicate that the sands likely were deposited in a nearshore beach or dune environment, possibly during the transgression and regression of the high sea level stand that formed the underlying marine terrace platform, which is interpreted to be middle to early Pleistocene in age (>340,000 years ago as discussed in FSAR Subsection 2.5.1.2.1.2). There may be a

component of younger eolian quartz sand deposited during subsequent sea level fluctuations and locally derived fluvial deposits. In some boreholes, thicker sections of the S1 deposits consist of gray quartz sand intermixed or interbedded with medium brown sand and grayish black clay and sandy clay layers. These deposits are interpreted to represent infills of sand and marsh deposits into paleosinks. Some of the infill material in the deeper paleosinks may be Tertiary as well as Quaternary in age.

The Quaternary sediments (unit S1) at the LNP site are differentiated from the top of the underlying calcareous silts (dolosilts, designated unit S2) at the top of the Avon Park Formation by their observed siliciclastic (quartz sand or other silicates) lithology, their typically gray to brown color, an absence of fossils, their subrounded to rounded grain shapes, and their lack of reaction to hydrochloric acid (HCL). The thickness of these Quaternary sediments varies across the LNP site from less than 3 m (10 ft.) to approximately 30 m (100 ft.), with a thickness of approximately 2 m (6 ft.) under the nuclear island. At a few boring locations, the thickness of the Quaternary sediments was higher, and the maximum thickness on site was measured at 73.5 m (241 ft.) in one boring completed as part of the pre-COLA siting investigations (Borehole NB 5), located beyond the perimeter of the LNP 2 site (Figure 2.5.1-251 and Figure 2.5.1-252). Based upon available boring data at the LNP 2 site, this local thickening of Quaternary sediments may represent infilling of localized paleokarst features or paleochannels.

The Avon Park Formation is of middle Eocene age, and is characterized as cream to brown or tan, poorly indurated to well-indurated, variably fossiliferous limestone, interbedded in places with tan to brown, very poorly to well-indurated, fossiliferous, vuggy dolostones. Carbonized plant remains are common in the rock sequence in the form of thin, poorly indurated laminae and cyclic interbeds. (Reference 2.5.1-204)

The Avon Park Formation is a carbonate mud-dominated peritidal depositional sequence, pervasively dolomitized in places and not dolomitized in others, that contains some intergranular and interbedded evaporites in its lower part. Fossils are mostly benthic forms showing limited faunal diversity. Seagrass beds are well preserved as lignite lenses at certain horizons. The lower portion of the Avon Park Formation consists of lower permeability evaporite deposits, which act as an aquitard separating the Upper Floridan aquifer within the Avon Park Formation from the Lower Floridan aquifer within the Oldsmar Limestone. (Reference 2.5.1-326)

Borings at the LNP site characterized the upper 150+ m (500 ft.) of the Avon Park Formation as consisting primarily of dolomitized limestone and dolostone. Limestone was encountered but was not widespread over the site. Much of the carbonate in the Avon Park Formation in west-central Florida was deposited as limestone (Reference 2.5.1-231). Dolomitization of the limestone occurred in the Oligocene (Reference 2.5.1-231).

At the LNP site, the Avon Park Formation occurs as a fossiliferous soft limestone near the top of the sequence, with evidence of increasing dolomitization and

recrystallization with depth, especially in a more dense rock zone at depths around 40 – 60 m (140 – 190 ft.) The evidence of marine fossils in site rock cores is primarily from casts and molds, which enhance rock porosity; whereas most original calcium carbonate fossil material has been dissolved away by groundwater dissolution. Lignite laminae and interbeds are common near the top of the sequence, and again at depths of approximately 120 m (400 ft.), where two lignite beds, each less than 0.3 m (1 ft.) thick, were observed site-wide. In many borings, the rock sequence becomes softer and less well-indurated limestone with poor core recovery at depths below approximately 61 m (200 ft.)

2.5.1.2.5.3 Site Location Karst Development

To evaluate the LNP site for karst potential, a variety of data sources were employed, including historical and recent aerial photos, published lineament analyses, topographic maps derived from LIDAR data, boring logs, core photos, surface geophysical testing, downhole geophysical logging, and downhole seismic testing.

The low-relief, relatively flat surface topography at the LNP site and surrounding area is characterized by circular to irregularly shaped shallow depressions of varying size. The irregular-shaped depressions are typical of karstic depressions observed elsewhere in the study region that are interpreted to result from coalescing smaller and shallow depressions (Reference 2.5.1-235). Rectilinear margins that define the edges of some of the topographic lows, orientations of the major axis of the depressions and associated wetlands, and alignments of many of the deeper circular features suggest that the location of the features is influenced by joint systems in the underlying rock (see FSAR Subsection 2.5.3.2.1.3).

Observations from the CR3 excavation indicated that solutioning occurred along fractures (joints), and in particular at bedding plane-fracture intersections, forming a network of essentially vertical solution channels that have been secondarily infilled with very fine quartz sands, organic silts and clays, and shells. The irregular surface of the top of the Avon Park Formation at the CR3 plant, which exhibits relief of approximately 10 m (30 ft.), was attributed to formation of karrenfield topography during a period of surface exposure, represented by the Jackson-Claiborne Unconformity, which marks the boundary between the Avon Park Formation and overlying Ocala Limestone. Thus, it is likely that some of the karst development observed in the top of the Avon Park Formation occurred during the late Eocene (approximately 40 million years ago).

The stratigraphy encountered at the CR3 site, which consists of Ocala Limestone over Avon Park Formation, is different from the LNP site where the Ocala Limestone is not present. The Avon Park Formation limestones typically exhibit higher degrees of dolomitization than the Upper Eocene Ocala Limestone. This is significant because the more dolomitized Avon Park Formation limestones have a higher percentage of recrystallized magnesium carbonate, and would therefore typically be less susceptible to the types of karst activity known to occur within

the pure calcium carbonate limestone zones present at the top of the Ocala Formation.

Subsurface data from the LNP site investigation indicate that there is variability in the elevation of the Quaternary/Tertiary unconformity and the contact between the S3 and AV1 units at depth. This topography likely reflects a variety of processes, including: 1) weathering and dissolution related to heterogeneities within the underlying carbonate rocks that are due to variable degrees of dolomitization or initial depositional properties; 2) planation and erosion related to Neogene or Quaternary marine transgressions/regressions; and 3) location and degree of development of the paleo-epikarstic surface that likely formed in the upper strata of the Avon Park Formation over a period of as much as several million years.

The LNP site stratigraphy and surface morphology are consistent with expected characteristics of a developed paleokarst landscape mantled by several meters of sand (i.e., a mantled epikarst subsurface) (Figure 2.5.1-244). There are no recognized sinkholes in the State of Florida sinkhole database within 2 km (1.28 mi.) of the LNP site (Figure 2.5.1-244), and no sinkholes at the land surface were observed during site investigations and reconnaissance within the LNP site. Site borings revealed very few voids in the upper 150+ m (500 ft.).

Although subsurface data from exploration boreholes at the LNP site indicate that there is variability in the elevation of the Q/T unconformity at the LNP site, the Q/T unconformity is generally at an elevation of 10.7 ± 0.6 m (35 ± 2 ft.) NAVD88 under the nuclear islands. It is assumed that this represents the general elevation of the marine planation surface, which is estimated to be older than MIS 9 (340 ka) and most likely middle to early Pleistocene, or possibly late Pliocene in age (FSAR Subsection 2.5.1.2.1.2). The nature, frequency, thickness, and lateral extent of subsurface karst features identified in borings under the safety-related structures are described in FSAR Subsection 2.5.4.1.2.1. These features generally vary in lateral extent from a few centimeters to approximately 1.5 m (5 ft.) when associated with vertical fracturing, and from a few centimeters to approximately 3.0 m (10 ft.) when associated with horizontal bedding planes.

2.5.1.2.6 Site Area Geologic Hazard Evaluation

Evaluation of geologic hazards at the LNP site was based on the compilation and review of published maps and reports, reconnaissance investigations in the site area, discussions with FGS and SWFWMD personnel and karst experts, and results of the site characterization program.

- The LNP site is located in an area of infrequent and low seismicity. Earthquake activity with resulting ground motion effects are considered in the seismic design ground motions for the site (see FSAR Subsection 2.5.2). There are no capable tectonic sources in the site area; thus, there is negligible potential for surface tectonic deformation at the site (see FSAR Subsection 2.5.3).

RAI Response

NRC Letter No.: ML0827602222

NRC Letter Date: October 6, 2008

NRC Review of Final Safety Analysis Report

NRC RAI #: 02.05.04-1

Text of NRC RAI:

Please provide a sufficiently detailed discussion to justify that the borings adequately characterize karst at depth at the site, and that the existing borehole spacing is sufficient to characterize the lateral dimension of dissolution cavities and assess their correlation and interpreted lack of connectivity between boreholes.

PGN RAI ID #:

PGN Response to NRC RAI:

The potential for karst features at depth is significantly reduced by two factors: (1) the nature of the karst features at LNP, and (2) the resistance of the Avon Park Formation to undergo further dissolution.

Regarding the nature of the karst features, those in the LNP vicinity are characterized by solution channels in limestone that are oriented along near-vertical fractures having trends of fracture systems mapped at the surface. Cavities are developed as the walls of fractures are dissolved away by recently recharged groundwater with high carbon dioxide content (Faulkner, 1973). Because groundwater percolates downward and carbon dioxide content decreases as the ground water percolates, the groundwater has reduced potential to dissolve limestone. Therefore, the size of potential karst features diminishes with depth.

Regarding the ability of the subsurface to undergo further dissolution, the Avon Park Formation is highly dolomitized, meaning that magnesium carbonate has replaced a significant percentage of calcium carbonate within the limestone. Eighteen of twenty samples taken during the LNP site characterization that were petrographically examined were completely dolomitized (less than 50% $CaCO_3$). From Easterbrook (1999), "*[the] purer the limestone is in $CaCO_3$, the greater is its proclivity to form karst. Some evidence suggests that about 60 percent $CaCO_3$ is necessary to form karst, and about 90 percent may be necessary to fully develop karst.*" Also, dolomites have a lower permeability rate than limestone, thereby reducing the occurrence of karst formation (Easterbrook, 1999).

In addition, dolomitized limestones have a smaller lattice structure than non-dolomitized limestone due to the size of the magnesium and calcite molecules. This smaller lattice reduces the volume of voids within the structure, which will reduce the amount of secondary porosity and fracturing within the rock. The smaller lattice of the dolomite is also crystallized, providing a stronger, compact lattice (Prothero et al, 1996).

The guidance of RG 1.132, Section 4.3.1.1, states that "[it] is important that…borings penetrate all suspect zones or extend to depths below which their presence would not influence the safety

of structures." As described in the supplement, karst features at depth (below El. -99 ft.) have been shown to not influence the safety of the structures (also see response to RAI 02.05.01-7).

Data from the boreholes that penetrate below El. -99 ft show no inconsistency with the analysis that shows no safety significant potential karst features at depth. Consistent with RG 1.132, borehole spacing that was sufficiently tight to characterize the lateral dimension of dissolution cavities and assess the connectivity between boreholes is not needed. Given the nature of the karst features at LNP and their reduced ability to undergo further dissolution, as well as the impact of such features on the safety of the structures, the characterization of these features as presented in the FSAR and the supplement is adequate.

References:

1) Easterbrook, D.J., *Surface Processes and Landforms*, Second Edition, Prentice Hall, Upper Saddle River, NJ, 1999.

2) Faulkner, G.L., *Geohydrology of the Cross-Florida Barge Canal Area with Special Reference to the Ocala Vicinity*, U.S. Geological Survey, Water-Resources Investigation Report I-73, 1973.

3) Prothero, D.R., Schwab, F., *Sedimentary Geology – An Introduction to Sedimentary Rocks and Stratigraphy*, First Edition, W.H. Freeman and Company, New York, 1996.

Associated LNP COL Application Revisions:

NOTE: (1) Include current COLA wording and revised wording. (2) Identify which changes (if any) are due to COLA preparation errors.

- Consider other impacted portions of the COLA in addition to FSAR, for example the ER

No COLA revisions have been identified associated with this response.

Attachments/Enclosures:

NOTE: Provide list of attachments/enclosures or state "No COLA revisions have been identified associated with this response" and attach files (if applicable). Ensure attachments pass pre-flight and are acceptable for submittal to NRC, or provide list of exceptions.

1) Easterbrook, D.J., *Surface Processes and Landforms*, Second Edition, Prentice Hall, Upper Saddle River, NJ, 1999.

2) Prothero, D.R., Schwab, F., *Sedimentary Geology – An Introduction to Sedimentary Rocks and Stratigraphy*, First Edition, W.H. Freeman and Company, New York, 1996.

3) Validation Package (for internal use)

RAI Response

NRC Letter No.: ML0827602222

NRC Letter Date: October 6, 2008

NRC Review of Final Safety Analysis Report

NRC RAI #: 02.05.01-1

Text of NRC RAI:

Please summarize the information being used as the technical basis for the dissolution rates presented, including documentation of the basis for indicating that dolomitized limestone dissolves less readily than non-dolomitized limestone, to enable an adequate assessment of karst development as a potential future geologic hazard. Include any references necessary.

PGN RAI ID #:

PGN Response to NRC RAI:

The technical basis for the anticipated dissolution rate is as follows:

The dissolution rate from the Crystal River 3 (CR3) site was used for assessing the dissolution rate at LNP. The rate associated with CR3 is presented in the Crystal River 3 FSAR Subsection 2.5.3.4. The dissolution rate at CR3 is calculated as 1E-4 percent per year, or 6E-3 percent over the projected 60-year life of the project.

Since CR3 is founded on the Ocala Formation, and LNP is to be founded on the Avon Park Formation, the second part of the technical basis was the comparison between those two limestone formations. The Avon Park Formation is more dolomitized than the Ocala Formation, meaning that magnesium carbonate has replaced a significant percentage of calcium carbonate within the limestone. Eighteen of twenty samples taken during the LNP site characterization that were petrographically examined were completely dolomitized (less than 50 percent $CaCO_3$). From Easterbrook (1999, attached), "*[the] purer the limestone is in $CaCO_3$, the greater is its proclivity to form karst. Some evidence suggests that about 60 percent $CaCO_3$ is necessary to form karst, and about 90 percent may be necessary to fully develop karst.*" Also, dolomites have a lower permeability rate than limestone, thereby reducing the occurrence of karst formation (Easterbrook, 1999, attached).

Further to this point, dolomitized limestones have a smaller lattice structure than non-dolomitized limestone due to the relative sizes of the magnesium carbonate molecules versus the calcite molecules. This smaller lattice reduces the volume of voids within the structure, which will reduce the amount of secondary porosity within the rock. The smaller lattice of the dolomite is also crystallized, providing a stronger and more compact lattice (Prothero et al, 1996).

In summary, dissolution rates are documented for the Ocala Formation at the CR3 site as 1E-4 percent per year; therefore, it is concluded from the second part of the technical basis, presented above, that the rate of dissolution at the LNP site will be less than 1E-4 percent per year given the higher degree of dolomitization of the Avon Park Formation at LNP. Therefore,

the potential for future geologic hazards in the form of dissolution and karst formation, during the plant life of LNP, is not significant.

As discussed during the presentation to the Staff in January 2008, a monitoring program will be established for the LNP plant to confirm the dissolution rate identified in this analysis. This monitoring is part of the groundwater monitoring program described in FSAR Subsection 2.4.12.4.

References:

1) Easterbrook, D.J., *Surface Processes and Landforms*, Second Edition, Prentice Hall, Upper Saddle River, NJ, 1999.

2) Prothero, D.R., Schwab, F., *Sedimentary Geology – An Introduction to Sedimentary Rocks and Stratigraphy*, First Edition, W.H. Freeman and Company, New York, 1996.

Associated LNP COL Application Revisions:

NOTE: (1) Include current COLA wording and revised wording. (2) Identify which changes (if any) are due to COLA preparation errors.

- Consider other impacted portions of the COLA in addition to FSAR, for example the ER

FSAR Subsection 2.5.4.12 will be modified to provide reference to the Operational Monitoring Program in FSAR Subsection 2.4.12.4.

Attachments/Enclosures:

NOTE: Provide list of attachments/enclosures or state "No COLA revisions have been identified associated with this response" and attach files (if applicable). Ensure attachments pass pre-flight and are acceptable for submittal to NRC, or provide list of exceptions.

1) Easterbrook, D.J., *Surface Processes and Landforms*, Second Edition, Prentice Hall, Upper Saddle River, NJ, 1999.

2) Prothero, D.R., Schwab, F., *Sedimentary Geology – An Introduction to Sedimentary Rocks and Stratigraphy*, First Edition, W.H. Freeman and Company, New York, 1996.

3) Validation Package (for internal use)

RAI Response

NRC Letter No.: ML0827602222

NRC Letter Date: October 6, 2008

NRC Review of Final Safety Analysis Report

NRC RAI #: 02.05.01-3

Text of NRC RAI:

The supplement states that grouting will inhibit the development of karst by preventing the flow of groundwater through the grouted zones beneath the nuclear island (Attachment 2, pg. 15 of supplement, Permeation Grouting Discussion).

Please address the potential issue of how altering the groundwater flow regime by grouting could affect dissolution below and around the periphery of the grouted zone to assure that this aspect has been considered.

PGN RAI ID #:

PGN Response to NRC RAI:

The uppermost 75 feet of the Avon Park Formation will be grouted to provide a flow barrier (i.e., bathtub) to allow for dewatering during NI construction. In addition, a 35-foot roller compacted concrete bridging mat will be installed above the grouted zone. The reduction in subsurface permeability associated with these construction activities, has been evaluated regarding the impact to potential dissolution and erosion at LNP.

In a three-dimensional groundwater model, the pre-grouting site conditions were simulated in steady-state flow conditions. A three-dimensional model was also created to determine the influence of grouted zone on the local groundwater regime. A three-dimensional model which simulated construction dewatering was also created. Groundwater heads, velocities and directions were compared under existing, construction, and post-construction scenarios.

All three computer models (i.e., pre-construction, construction, and post-construction) contain 9 virtual observation wells, located around the NI in which the hydraulic heads and groundwater velocities can be reported. One virtual well was placed at the center of the NI area. Two virtual wells were placed at all four sides of the NI area (e.g., E1 and E2 at the east side as observation wells East 1 and East 2), placed 12.5 ft and 62.5 ft respectively from the future diaphragm wall location. With two virtual observation points at each side of the NI, the pattern in the hydraulic heads and gradients around the NI at these observation wells was assessed. Hydraulic heads and groundwater velocities in three dimensions were determined in two subsurface zones: Zone A is designated as the zone between elevations -24 ft and -99 ft; Zone B is designated as the zone between elevations -99 ft. and -250 ft. Zone A addresses the groundwater flow regime around the grouted zone, and Zone B addresses the groundwater flow regime beneath the grouted zone.

Groundwater parameters used in the analysis are consistent with information presented in FSAR Subsection 2.4.12.

The hydraulic heads observed in the pre-construction and post-construction models are equivalent, as expected due to the presence of the diaphragm wall and grouted zone. In the post-construction model, the maximum groundwater velocity is in Zone B at observation well E1 as 0.16 ft/day. The maximum increase in the groundwater velocity between pre- and post-construction conditions is approximately 40 percent.

The change in the groundwater regime in the post-construction condition is judged to be insignificant with regards to increasing the potential for karst development within the Avon Park Formation, for two primary reasons:

1) An analysis considering the normal forces on a soil particle (i.e., drag force and shelf weight of the particle) has been performed. The results indicate that a groundwater velocity of 0.26 ft/day is necessary to transport the smallest (1 micron) clay particle. In a more advanced evaluation of the normal and tangential forces on a soil particle, including electrical attraction forces, it was shown (Santamarina, 2001) that a minimum of 283 ft/day groundwater velocity is necessary to cause a detachment of any particle. In the post-construction model, a maximum groundwater velocity of 0.16 ft/day was determined. This increase in velocity will not allow for the transport of fines and subsequent creation of cavities, particularly since the infilled zones at LNP mostly contain larger silt- and sand-sized material. The increase in groundwater velocity due to LNP construction is not large enough to move the particles of any infilled zones, and therefore additional erosion activity is negligible.

2) Given the dissolution rate at the Crystal River 3 plant (see response to RAI 02.05.01-1), the dissolution rate at LNP is estimated to be negligible. If this rate were increased by 40 percent, taking the conservative assumption that an increase in the groundwater velocity is linearly proportional to an increase in dissolution, the predicted dissolution rate would still be negligible.

The hydraulic heads observed outside of the diaphragm wall in the construction (dewatering) model are within 0.7 feet of the pre-construction model. This temporary increase in hydraulic gradient has been considered in the determination of increased groundwater velocities.

During the dewatering of the LNP subsurface for construction, the change in groundwater regime is judged to be insignificant with regards to increasing the potential for karst development within the Avon Park Formation, for two primary reasons:

1) In the construction model, a maximum groundwater velocity of 1.14 ft/day was determined. The maximum particle diameter that can be removed with this maximum velocity is 2 microns, or a small clay particle. Since the infilled zones at LNP mostly contain larger silt- and sand-sized material, the increase in groundwater velocity due to LNP construction is not large enough to move the particles of any infilled zones, and therefore additional erosion activity is negligible.

2) Since the construction period will occur over a geologically short, discrete amount of time, dissolution of the Avon Park Formation during this time period is negligible.

Based on the above, the impacts of LNP dewatering and permanent construction on the existing groundwater regime have been considered, and have been shown to be negligible with respect to an increased development of dissolution and erosion activity.

As discussed during the presentation to the Staff in January 2008, a monitoring program will be established for the LNP plant to confirm the dissolution rate identified in this analysis. This monitoring is part of the groundwater monitoring program described in FSAR Subsection 2.4.12.4.

References:

Santamarina, J.C., *Soil Behavior at the Microscale: Particle Forces*, Proc. Symp. Soil Behavior and Soft Ground Construction, October 2001.

Associated LNP COL Application Revisions:

NOTE: (1) Include current COLA wording and revised wording. (2) Identify which changes (if any) are due to COLA preparation errors.

- Consider other impacted portions of the COLA in addition to FSAR, for example the ER

FSAR Subsection 2.5.4.12 will be modified to provide reference to the Operational Monitoring Program in FSAR Subsection 2.4.12.4.

Attachments/Enclosures:

NOTE: Provide list of attachments/enclosures or state "No COLA revisions have been identified associated with this response" and attach files (if applicable). Ensure attachments pass pre-flight and are acceptable for submittal to NRC, or provide list of exceptions.

1) Santamarina, J.C., *Soil Behavior at the Microscale: Particle Forces*, Proc. Symp. Soil Behavior and Soft Ground Construction, October 2001.

2) Validation Package (for internal use)

RAI Response

NRC Letter No.: ML0827602222

NRC Letter Date: October 6, 2008

NRC Review of Final Safety Analysis Report

NRC RAI #: 02.05.01-5

Text of NRC RAI:

The supplement lists assumptions and postulations used to calculate lateral dimensions of borehole features (Attachment 2, pg. 7 of supplement, Karst Discussion - Excess Grout Takes), and states that 9.9 ft is the maximum lateral extent of dissolution cavities at depth. Considering a fracture spacing of 19 ft., if dissolution developed along two parallel fractures with this spacing, then the resulting cavity could easily exceed 9.9 ft. if the two cavities coalesced at depth.

Please discuss the uncertainty involved in the estimate of a 9.9 ft. maximum lateral extent for dissolution cavities and the potential for coalescing dissolution cavities at depth.

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The analysis for both karst size and potential coalescence is based on conservative parameters to account for the uncertainty associated with the observed data. Therefore, no differentiation is made between "conservatism" and "uncertainty." The analysis relies upon a margin as described below.

Several conservatisms were applied during the analysis that yielded the size of the design karst feature. As described in the supplement, the following conservatisms were employed:

1) The excess grout volume was conservatively estimated by assuming that each borehole was only 3.25 inches in diameter, from ground surface to the termination depth. The "excess grout" calculated was the grout that remained after subtracting the volume in the 3.25 inch diameter borehole from the total grout.

2) The excess grout was distributed to only certain potential karst features identified within each borehole.

3) For features associated with vertical fractures, the excess grout volume was conservatively increased by 50 percent; for features associated with bedding planes, the excess grout volume was conservatively increased by 100 percent.

With those conservatisms, the average lateral extent associated with a vertical feature was 3.1 feet, and the maximum was 6.1 feet. The average lateral extent associated with a horizontal feature was 6.5 feet, and the maximum was 9.9 feet.

To better demonstrate the conservatism associated with the size of the design void, the data were re-analyzed using the following alternate assumptions:

1) The excess grout volume was conservatively estimated by assuming that each borehole was only 3.25 inches in diameter, from ground surface to the termination depth. The "excess grout" calculated was the grout that remained after subtracting the volume in the 3.25 inch diameter borehole from the total grout. (*Same assumption as the original analysis.*)

2) Excess grout is distributed over all low density features identified within each borehole.

3) Excess grout volume is defined as only the grout exceeding the theoretical borehole size (no conservative increases to the grout volume).

With these less conservative assumptions, the average lateral extent associated with a vertical feature is 1.8 feet, and the maximum is 4.3 feet. The average lateral extent associated with a horizontal feature is 3.5 feet, and the maximum is 5.3 feet.

Therefore, the design void is larger than the calculated maximum void by a factor of 1.9 (10 feet versus 5.3 feet).

If two 10-foot voids were to develop at adjacent local fractures (the worst-case scenario), the features would be separated by approximately 9 feet of undissolutioned Avon Park Formation, since the "plus-sign" morphology of the karst development in this region would govern that the void would extend 5 feet in each direction from the vertical fracture. The potential for coalescence of two adjacent dissolution cavities is diminished because of the dolomitization of the Avon Park Formation. Not only were effective inter-granular porosity and permeability reduced in the Avon Park Formation by recrystallization during dolomitization but, development of solution channel porosity and permeability from groundwater circulation also was reduced because dolomite is much less soluble than limestone (Faulkner, 1973).

While it is unlikely that features would coalesce at depth given the dolomitic nature of the Avon Park Formation, the effect of such coalescence on the RCC bridging mat has been evaluated. As shown on the supplement Figure 2.5.4.5-204, Case B-5, a 10-foot cavity that spans the width of the nuclear island has been included in an elastic stress analysis. Such a cavity, which effectively models a series of 10-foot voids that have coalesced across the entire nuclear island, is shown to be bridged by the RCC bridging mat without adverse effects on the safety of the plant.

References:

1) Faulkner, G.L., *Geohydrology of the Cross-Florida Barge Canal Area with Special Reference to the Ocala Vicinity*, U.S. Geological Survey, Water-Resources Investigation Report I-73, 1973.

Associated LNP COL Application Revisions:

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(2) Identify which changes (if any) are due to COLA preparation errors.

- Consider other impacted portions of the COLA in addition to FSAR, for example the ER

No COLA revisions have been identified associated with this response.

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Validation Package (for internal use)