2.0 SITE CHARACTERISTICS

Chapter 2, "Site Characteristics," of the Final Safety Analysis Report (FSAR) addresses the geological, seismological, hydrological, and meteorological characteristics of the site and vicinity, in conjunction with present and projected population distribution and land use, and site activities and controls.

2.0.1 Introduction

The site characteristics are reviewed by the Nuclear Regulatory Commission (NRC) staff to determine whether the applicant has accurately described the site characteristics and site parameters in accordance with Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52, "Licenses, certifications, and approvals for nuclear power plants." The review is focused on the site characteristics and site-related design characteristics needed to enable the NRC staff to reach a conclusion on all safety matters related to siting of the Levy Nuclear Plant (LNP) Units 1 and 2. Because this combined license (COL) application references a design certification (DC), this section focuses on the applicant's demonstration that the characteristics of the site fall within the site parameters specified in the DC rule or, if outside the site parameters, that the design satisfies the requirements imposed by the specific site characteristics and conforms to the design commitments and acceptance criteria described in the AP1000 Design Control Document (DCD).

2.0.2 Summary of Application

Section 2.0 of the LNP COL FSAR, Revision 9, incorporates by reference Chapter 2 of the AP1000 DCD, Revision 19. AP1000 DCD Chapter 2 includes Section 2.0 of the LNP COL FSAR.

In addition, in LNP COL FSAR Section 2.0, the applicant provided the following:

Supplemental Information

- LNP Supplemental (SUP) 2.0-1

The applicant provided supplemental information in LNP COL FSAR Section 2.0, "Site Characteristics," which describes the site characteristics of LNP. The applicant also provided LNP COL FSAR Table 2.0-201, which provides a comparison of the LNP site characteristics and the AP1000 DCD Site Parameters in AP1000 DCD Tier 1 Table 5.0-1 and DCD Tier 2 Table 2-1.

2.0.3 Regulatory Basis

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793, "Final Safety Evaluation Report Related to Certification of the AP1000 Standard Design" (September 2004), and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the site characteristics are given in Section 2.0 of NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)."

The applicable regulatory requirements for site characteristics are as follows:

- 10 CFR 52.79(a)(1)(i) - (vi) provides requirements for the site-related contents of the application.
- 10 CFR 52.79(d)(1), as it relates to information sufficient to demonstrate that the characteristics of the site fall within the site parameters specified in the DC.
- 10 CFR Part 100, "Reactor site criteria," as it relates to the siting factors and criteria for determining an acceptable site.

The related acceptance criteria from Section 2.0 of NUREG-0800 are as follows:

- The acceptance criteria associated with specific site characteristics/parameters and site-related design characteristics/parameters are addressed in the related Chapter 2 and Chapter 3 sections of NUREG-0800.
- Acceptance is based on the applicant's demonstration that the characteristics of the site fall within the site parameters of the certified design. If the actual site characteristics do not fall within the certified standard design site parameters, the COL applicant provides sufficient justification (e.g., by request for exemption or amendment from the DC) that the proposed facility is acceptable at the proposed site.

2.0.4 Technical Evaluation

The NRC staff reviewed Section 2.0 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to site characteristics. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

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[1] See Section 1.2.2 for a discussion of the staff's review related to verification of the scope of information to be included in a COL application that references a DC.

Supplemental Information

- LNP SUP 2.0-1

The NRC staff reviewed supplemental information LNP SUP 2.0-1 in LNP COL FSAR Section 2.0 describing the site characteristics of LNP Units 1 and 2. The AP1000 DCD site parameters in DCD Tier 2 Table 2-1 are compared to the site-specific site characteristics in LNP COL FSAR Table 2.0-201. In addition, control room atmospheric dispersion factors for accident dose analysis are presented in LNP COL FSAR Table 2.0-202.

The NRC staff reviewed and compared the site-specific characteristics included in LNP COL FSAR Table 2.0-201 against AP1000 DCD Tier 2 Table 2-1 and DCD Tier 1 Table 5.0-1. The staff's evaluation of the site characteristics associated with air temperature, precipitation, wind speed, atmospheric dispersion values, and control room atmospheric dispersion values is addressed in Section 2.3 of this Safety Evaluation Report (SER). The staff's evaluation of site characteristics associated with flood level, ground water level, and plant grade elevation is addressed in Section 2.4 of this SER. The staff's evaluation of seismic and soil site characteristics is addressed in Section 2.5 of this SER. The staff's evaluation of site characteristics associated with missiles is addressed in Section 3.5 of this SER.

The site-specific characteristics listed in LNP COL FSAR Table 2.0-201 are enveloped by the AP1000 DCD site parameter values addressed in DCD Tier 2 Table 2-1 and DCD Tier 1 Table 5.0-1.

2.0.5 Post Combined License Activities

There are no post-COL activities related to this section.

2.0.6 Conclusion

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to site characteristics, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the NRC staff reviewed the application to ensure that sufficient information was presented in LNP SUP 2.0-1 to demonstrate that the characteristics of the site fall within the site parameters specified in the DC. The applicant has demonstrated that the requirements of 10 CFR 52.79(d)(1) have been met.

2.1 Geography and Demography

2.1.1 Site Location and Description

2.1.1.1 *Introduction*

The descriptions of the site area and reactor location are used to assess the acceptability of the reactor site. The review covers the following specific areas: (1) specification of reactor location with respect to latitude and longitude, political subdivisions; and prominent natural and manmade features of the area; (2) site area map to determine the distance from the reactor to the boundary lines of the exclusion area, including consideration of the location, distance, and orientation of plant structures with respect to highways, railroads, and waterways that traverse or lie adjacent to the exclusion area; and (3) any additional information requirements prescribed in the “Contents of Application” sections of the applicable subparts to 10 CFR Part 52. The purpose of the review is to ascertain the accuracy of the applicant’s description for use in independent evaluations of the exclusion area authority and control, the surrounding population, and nearby manmade hazards.

2.1.1.2 *Summary of Application*

Section 2.1 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.1 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.1, the applicant provided the following:

Tier 2 Departure

- STD DEP 1.1-1

The applicant proposed the following Tier 2 standard (STD) departure (DEP) from the AP1000 DCD. Part 7 of the LNP application identifies instances where the FSAR sections are renumbered to include content consistent with Regulatory Guide (RG) 1.206, “Combined License Applications for Nuclear Power Plants (LWR Edition),” as well as NUREG-0800 rather than following the AP1000 DCD numbering. In addition, LNP Part 7 requests an exemption from the requirement to use the same organization and numbering as the AP1000 DCD. In LNP COL FSAR Section 2.1, “Geography and Demography,” Section 2.1.1 of the AP1000 DCD is renumbered as Section 2.1.4.

AP1000 COL Information Item

- LNP COL 2.1-1

The applicant provided additional information in LNP COL 2.1-1 to resolve COL Information Item 2.1-1 (COL Action Item 2.1.1-1), which addresses the provision of site-specific information related to site location and description, including political subdivisions, natural and man-made features, population, highways, railways, waterways, and other significant features of the area.

2.1.1.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the site location and description are given in Section 2.1.1 of NUREG-0800.

The applicable regulatory requirements for identifying site location and description are:

- 10 CFR 50.34(a)(1) and 10 CFR 52.79(a)(1), as they relate to the inclusion in the safety analysis report (SAR) of a detailed description and safety assessment of the site on which the facility is to be located, with appropriate attention to features affecting facility design.

- 10 CFR Part 100, as it relates to the following: (1) defining an exclusion area and setting forth requirements regarding activities in that area (10 CFR 100.3); (2) addressing and evaluating factors that are used in determining the acceptability of the site as identified in 10 CFR 100.20(b); (3) determining an exclusion area such that certain dose limits would not be exceeded in the event of a postulated fission product release as identified in 10 CFR 50.34(a)(1), as it relates to site evaluation factors identified in 10 CFR Part 100; and (4) requiring that the site location and the engineered features included as safeguards against the hazardous consequences of an accident, should one occur, would ensure a low risk of public exposure. In particular, 10 CFR 100.20(a), and 10 CFR 100.21 require that population density and use characteristics of the site environs, including the exclusion area, low-population zone, and population center distance, be considered in determining the acceptability of a site for a stationary power reactor.

The related acceptance criteria from Section 2.1.1 of NUREG-0800 are as follows:

- Specification of Location: The information submitted by the applicant is adequate and meets the requirements of 10 CFR 50.34(a)(1) and 10 CFR 52.79(a)(1) if it describes highways, railroads, and waterways that traverse the exclusion area in sufficient detail to allow the reviewer to determine that the applicant has met the requirements in 10 CFR 100.3.

- Site Area Map: The information submitted by the applicant is adequate and meets the requirements of 10 CFR 50.34(a)(1) and 10 CFR 52.79(a)(1) if it describes the site location, including the exclusion area and the location of the plant within the area, in sufficient detail to enable the reviewer to evaluate the applicant's analysis of a postulated fission product release, thereby allowing the reviewer to determine (in SER Sections 2.1.2 and 2.1.3, and Chapter 15) that the applicant has met the requirements of 10 CFR 50.34(a)(1) and 10 CFR Part 100.

The regulatory requirement associated with the Tier 2 departure request is as follows:

- 10 CFR Part 52, “Licenses, certifications, and approvals for nuclear power plants,” Appendix D, “Design Certification Rule for the AP1000 Design,” Section VIII, “Processes for Changes and Departures,” Item B.5.

2.1.1.4 *Technical Evaluation*

The NRC staff reviewed Section 2.1 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff’s review confirmed that the information in the application and incorporated by reference addresses the required information relating to the site location and description. The results of the NRC staff’s evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

Tier 2 Departure

- STD DEP 1.1-1

The applicant’s evaluation in accordance with Item B.5 of Section VIII of Appendix D to 10 CFR Part 52 determined that this departure did not require prior NRC approval. The NRC staff finds that it is reasonable that the departure does not require prior NRC approval because the numbering system proposed by the applicant does not alter the information required to be provided. A detailed evaluation of STD DEP 1.1-1 and the associated exemption can be found in Section 1.5.4 of this SER.

AP1000 COL Information Item

- LNP COL 2.1-1

The NRC staff reviewed LNP COL 2.1-1 related to site location and description, including political subdivisions, natural and man-made features, population, highways, railways, waterways, and other significant features of the area included in Section 2.1.1 of the LNP COL FSAR. COL Information Item 2.1-1 in Section 2.1.1 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will provide site-specific information related to site location and description, exclusion area authority and control, and population distribution. Site-specific information on the site and its location will include political subdivisions, natural and man-made features, population, highways, railways, waterways, and other significant features of the area.

The NRC staff, using publicly available maps, has independently verified the latitude and longitude supplied by the applicant. The NRC staff then converted this latitude and longitude to

Universal Transverse Mercator (UTM) coordinates for the proposed LNP Units 1 and 2 and used the calculated values to verify the UTM coordinates provided in the FSAR.

The NRC staff reviewed the site area map provided in the FSAR for the proposed Units 1 and 2 to verify that the distance from the reactor to the boundary line of the exclusion area meets the guidance in NUREG-0800 Section 2.1.1. On the basis of the NRC staff's review of the information in the LNP COL FSAR, and also the NRC staff's confirmatory review of the political subdivisions, and prominent natural and manmade features of the area as described in publically available documentation, the NRC staff determined the information provided by the applicant with regard to the site location and description is considered adequate and acceptable.

2.1.1.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.1.1.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to site location and description, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the applicant has presented and substantiated information to establish the site location and description. The staff has reviewed LNP COL 2.1-1, and for the reasons given in Section 2.1.1.4, concludes that it is sufficient for the staff to evaluate compliance with the siting evaluation factors in 10 CFR 100.3, as well as with the radiological consequence evaluation factors in 10 CFR 52.79(a)(1). The staff further concludes that the applicant provided sufficient details about the site location and site description to allow the staff to evaluate, as documented in Sections 2.1.2, 2.1.3, and 13.3 and Chapters 11 and 15 of this SER, whether the applicant has met the relevant requirements of 10 CFR Part 52.79(a)(1) and 10 CFR Part 100 with respect to determining the acceptability of the site.

The staff also concluded that STD DEP 1.1-1 meets the requirements for departures in 10 CFR Part 52, Appendix D, Section VIII, Item B5 and is, therefore, acceptable.

2.1.2 Exclusion Area Authority and Control

2.1.2.1 *Introduction*

The applicant's descriptions of exclusion area authority and control, which are used to verify the applicant's legal authority to determine and control activities within the designated exclusion area, are sufficient to enable the NRC staff to assess the acceptability of the reactor site. This review covers the following specific areas: (1) the applicant establishes its legal authority to determine all activities within the designated exclusion area, (2) the applicant establishes authority and control in excluding or removing personnel and property in the event of an emergency, (3) the applicant establishes that proposed or permitted activities in the exclusion area unrelated to operation of the reactor do not result in a significant hazard to public health and safety, and (4) the applicant provides additional information requirements as prescribed within the "Contents of Application" sections of the applicable Subparts to 10 CFR Part 52.

2.1.2.2 *Summary of Application*

Section 2.1 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.1 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.1.2, the applicant provided the following:

AP1000 COL Information Item

- LNP COL 2.1-1

The applicant provided additional information in LNP COL 2.1-1 to resolve COL Information Item 2.1-1 (COL Action Item 2.1.2-1), which addresses the provision of site-specific information related to exclusion area authority and control, including size of the area, exclusion area authority and control, and activities that may be permitted within the designated exclusion area.

2.1.2.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the exclusion area authority and control are given in Section 2.1.2 of NUREG-0800.

The applicable regulatory requirements for verifying exclusion area authority and control are:

- 10 CFR 50.34(a)(1), and 10 CFR 52.79(a)(1), as these regulations relate to the inclusion in the SAR of a detailed description and safety assessment of the site on which the facility is to be located, with appropriate attention to features affecting facility design (10 CFR 50.34(a)(1), and 10 CFR 52.79(a)(1)).

- 10 CFR Part 100, as it relates to the following: (1) defining an exclusion area and setting forth requirements regarding activities in that area (10 CFR 100.3); (2) addressing and evaluating factors that are used in determining the acceptability of the site as identified in 10 CFR 100.20(b); and (3) determining an exclusion area such that certain dose limits would not be exceeded in the event of a postulated fission product release as identified in 10 CFR 50.34(a)(1) as it relates to site evaluation factors identified in 10 CFR Part 100.

The related acceptance criteria from Section 2.1.2 of NUREG-0800 are as follows:

- Establishment of Authority for the Exclusion or Removal of Personnel and Property: The information submitted by the applicant is adequate and meets the requirements of 10 CFR 50.33, 10 CFR 50.34(a)(1), 10 CFR 52.79, and 10 CFR Part 100 if it provides sufficient detail to enable the staff to evaluate the applicant's legal authority for the exclusion or removal of personnel or property from the exclusion area.

- Proposed and Permitted Activities: The information submitted by the applicant is adequate and meets the requirements of 10 CFR 50.33, 10 CFR 50.34(a)(1), 10 CFR 52.79, and 10 CFR Part 100 if it provides sufficient detail to enable the staff to evaluate the applicant's legal authority over all activities within the designated exclusion area.

2.1.2.4 *Technical Evaluation*

The NRC staff reviewed Section 2.1.2 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the exclusion area authority and control. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

AP1000 COL Information Item

- LNP COL 2.1-1

The NRC staff reviewed LNP COL 2.1-1 related to the exclusion area authority and control, including size of the area, exclusion area authority and control, and activities that may be permitted within the designated exclusion area included in Section 2.1.2 of the LNP COL FSAR. COL Information Item in Section 2.1.1 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will provide site-specific information related to site location and description, exclusion area authority and control, and population distribution. Site-specific information on the exclusion area will include the size of the area and the exclusion area authority and control. Activity that may be permitted within the exclusion area will be included in the discussion.

The applicant supplied the following information: There are no residences, unauthorized commercial activities, or recreational activities within the Units 1 and 2 exclusion area. No public highways or active railroads not owned and controlled by the applicant traverse the exclusion area. There are no residents in the exclusion area. No unrestricted areas within the site boundary area are accessible to members of the public. The acceptance criteria for NUREG-0800, Section 2.1.2 state that, "Absolute ownership of all lands, including mineral rights, is considered to carry with it the required authority to determine all activities on this land and is acceptable." The NRC staff verified that the applicant owns all of the land in the exclusion area and site boundary, including mineral rights.

The NRC staff also verified for consistency that the exclusion area boundary (EAB) is the same as being considered for the radiological consequences in Chapter 15 and Section 13.3 of the FSAR by the applicant. The acceptance criteria of NUREG-0800, Section 2.1.2 states "Absolute ownership of all lands within the exclusion area, including mineral rights, is considered to carry with it the required authority to determine all activities on this land and is acceptable." Thus, the staff concludes that the applicant has the required authority to control all activities within the designated exclusion area.

The NRC staff used publically available maps, satellite pictures, and the area map provided in the Unit 1 and 2 FSAR to verify that no publicly used transportation mode crosses the EAB; therefore, arrangements for the control of traffic in the event of an emergency are not required.

The NRC staff, using maps, satellite pictures and the area map provided in the Unit 1 and 2 FSAR verified that no public roads cross the exclusion area; therefore, neither relocation nor abandonment of roads is needed.

2.1.2.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.1.2.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to the exclusion area authority and control, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the applicant has provided and substantiated information concerning its legal authority and control of all activities within the designated exclusion area. The staff has reviewed LNP COL 2.1-1, and for the reasons given above, concludes that the applicant's exclusion area is acceptable to meet the requirements of 10 CFR 50.34(a)(1), 10 CFR 52.79(a)(1), 10 CFR Part 100, and 10 CFR 100.3. This conclusion is based on the applicant having appropriately described the plant exclusion area, the authority under which all activities within the exclusion area can be controlled, the methods by which the relocation or abandonment of public roads that lie within the proposed exclusion area can be accomplished, if necessary, and the methods by which access and occupancy of the exclusion area can be controlled during normal operation and in the event of an emergency situation. In addition, the applicant has the required authority to control activities within the designated exclusion area, including the exclusion and removal of persons and property, and has established acceptable methods for control of the designated exclusion area.

2.1.3 Population Distribution

2.1.3.1 *Introduction*

The description of population distributions addresses the need for information about: (1) population in the site vicinity, including transient populations; (2) population in the exclusion area; (3) whether appropriate protective measures could be taken on behalf of the populace in the specified low-population zone (LPZ) in the event of a serious accident; (4) whether the nearest boundary of the closest population center containing 25,000 or more residents is at least one and one-third times the distance from the reactor to the outer boundary of the LPZ; (5) whether the population density in the site vicinity is consistent with the guidelines given in Regulatory Position C.4 of RG 4.7, "General Site Suitability Criteria for Nuclear Power Stations"; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.1.3.2 *Summary of Application*

Section 2.1 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.1 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.1.3, the applicant provided the following:

AP1000 COL Information Item

- LNP COL 2.1-1

The applicant provided additional information in LNP COL 2.1-1 to resolve COL Information Item 2.1-1 (COL Action Item 2.1.3-1), which addresses the provision of site-specific information related to population distribution for the site environs.

2.1.3.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for population distribution are given in Section 2.1.3 of NUREG-0800.

The applicable regulatory requirements for identifying site location and description are:

- 10 CFR 50.34(a)(1), as it relates to consideration of the site evaluation factors identified in 10 CFR 100.3, 10 CFR Part 100 (including consideration of population density), 10 CFR 52.79, as they relate to provision by the applicant in the SAR of the existing and projected future population profile of the area surrounding the site.

- 10 CFR 100.20 and 10 CFR 100.21, as they relate to determining the acceptability of a site for a power reactor. In 10 CFR 100.3, 10 CFR 100.20(a), and 10 CFR 100.21(b), the NRC provides definitions and other requirements for determining an exclusion area, LPZ, and population center distance.

The related acceptance criteria from Section 2.1.3 of NUREG-0800 are as follows:

- Population Data: The population data supplied by the applicant in the SAR is acceptable under the following conditions: (1) the FSAR includes population data from the latest census and projected population at the year of plant approval and 5 years thereafter, in the geographical format given in Section 2.1.3 of RG 1.70, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)," Revision 3, and in accordance with DG-1145; (2) the FSAR describes the methodology and sources used to obtain the population data, including the projections; and (3) the FSAR includes information on transient populations in the site vicinity.

- Exclusion Area: The exclusion area should either not have any residents, or such residents should be subject to ready removal if necessary.

- Low-Population Zone: The specified LPZ is acceptable if it is determined that appropriate protective measures could be taken on behalf of the enclosed populace in the event of a serious accident.

- Nearest Population Center Boundary: The nearest boundary of the closest population center containing 25,000 or more residents is at least one and one-third times the distance from the reactor to the outer boundary of the LPZ.

- Population Density: If the population density exceeds the guidelines given in Regulatory Position C.4 of RG 4.7, the applicant must give special attention to the consideration of alternative sites with lower population densities.

2.1.3.4 *Technical Evaluation*

The NRC staff reviewed Section 2.1.3 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to population distribution. The results of the NRC staff's evaluation of this information that is incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.1-1

The NRC staff reviewed LNP COL 2.1-1 related to the population distribution around the site environs included in Section 2.1.3 of the LNP COL FSAR. COL information item in Section 2.1.1 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will provide site-specific information related to site location and description, exclusion area authority and control, and population distribution. Site-specific information will be included on population distribution.

The staff reviewed the data on the population in the site environs, as presented in the applicant's FSAR, to determine whether the exclusion area, LPZ, and population center distance for the proposed LNP site comply with the requirements of 10 CFR Part 100. The staff also evaluated whether, consistent with Regulatory Position C.4 of RG 4.7 with regard to population density, the applicant should consider alternative sites with lower population densities. The staff also reviewed whether appropriate protective measures could be taken on behalf of the enclosed populace within the emergency planning zone (EPZ), which encompasses the LPZ, in the event of a serious accident. The LPZ consist of two circles each with a radius of 3 miles and centered on each of the LNP reactor buildings. The staff verified the applicant's population data against U.S. Census Bureau data. Transient population estimates were based on evaluations of seasonal transient business, hotels, motels, recreation, schools, hospitals, nursing homes, correctional facilities, festivals, and migrant worker populations. The staff reviewed and verified the projected population data provided by the applicant, including the weighted transient population for 2005, 2010, 2015, 2020, 2030, 2040, 2050, 2060, 2070, and 2080. Based on this information, the staff finds that the applicant's estimate of the population and population projections are reasonable.

The nearest population center to the LNP site with more than 25,000 residents is the city of Ocala, Florida, with a 2000 population of 45,622. The closest point of Ocala's corporate limit to the LNP site was determined to be approximately 30.1 miles to the east-northeast of the site. This distance is over ten times the distance from the center of Units 1 and 2 to the closest LPZ boundary. This distance meets the requirement that the population center distance is at least one and one-third times the distance from the reactor to the outer boundary of the LPZ as stipulated in 10 CFR 100.21(b). The NRC staff's review of population centers closer than Ocala did not identify any population centers that were projected to reach a population of 25,000 prior to the projected end of plant life. The distance factors described in NUREG-0800, Section 2.1.1 and RG 4.7 Section C.4 are met. Therefore, the NRC staff concludes that the proposed site meets the population center distance requirement in accordance with 10 CFR 100.21.

Regulatory Position C.4 of RG 4.7, Revision 2 states that the population density, including the weighted transient population projected at the time of initial site approval and 5 years thereafter should not exceed 500 persons per square mile averaged over any radial distance out to 20 miles (cumulative population at a distance divided by the area at that distance).

The NRC staff evaluated the site population density provided by the applicant in FSAR Table 2.1.3-207 against the guidance in Regulatory Position C.4 of RG 4.7, Revision 2. Table 2.1.3-207 indicates that the population density for the years 2000 through the year 2020 is between 97 and 146 persons per square mile. Therefore, the population density would not exceed 500 persons per square mile averaged over a radial distance of up to 20 miles (cumulative population at a distance divided by the area at that distance). The NRC staff independently verified these estimates by reviewing U.S. Census Bureau data and concludes that the population density is consistent with the demographic factors of RG 4.7, Revision 2. Therefore, the NRC staff concludes that the LNP application is consistent with Regulatory Position C.4 of RG 4.7, Revision 2.

2.1.3.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.1.3.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to population distribution, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the applicant has provided an acceptable description of current and projected population densities in and around the site. The staff has reviewed LNP COL 2.1-1, and for the reasons given above, concludes that the population data meets the requirements of 10 CFR 50.34(a)(1), 10 CFR 52.79(a)(1), and 10 CFR 100.20(a) and (b). The staff found that the applicant provided an acceptable description and safety assessment of the site, which includes present and projected population densities that are consistent with Regulatory Position C.4 of RG 4.7, and the applicant properly specified the LPZ and population center distance. In addition, the staff has reviewed and confirmed, by comparison with independently obtained U.S. Census Bureau population data, that the applicant's estimates of the present and projected populations surrounding the site, including transients, are accurate.

2.2 Nearby Industrial, Transportation, and Military Facilities

2.2.1 Locations and Routes

2.2.1.1 *Introduction*

The description of locations and routes refers to potential external hazards or hazardous materials that are present or may reasonably be expected to be present during the projected lifetime of the proposed plant. The purpose is to evaluate the sufficiency of information concerning the presence and magnitude of potential external hazards so that the reviews and evaluations described in NUREG-0800, Sections 2.2.3, 3.5.1.5, and 3.5.1.6 can be performed. The review covers the following specific areas: (1) the locations of, and separation distances to, transportation facilities and routes, including airports and airways, roadways, railways, pipelines, and navigable bodies of water; (2) the presence of military and industrial facilities, such as fixed manufacturing, processing, and storage facilities; and (3) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

The NRC staff's review of LNP COL FSAR Section 2.2.1, "Locations and Routes," and Section 2.2.2, "Descriptions," is addressed in this SER section.

2.2.1.2 *Summary of Application*

Section 2.2 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.2 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.2, the applicant provided the following:

Tier 2 Departure

- STD DEP 1.1-1

The applicant proposed the following Tier 2 departure from the AP1000 DCD. Part 7 of the LNP application identifies instances where the FSAR sections are renumbered to include content consistent with RG 1.206, as well as NUREG-0800. In addition, LNP Part 7 requests an exemption from the requirement to use the same organization and numbering as the AP1000 DCD. In LNP COL FSAR Section 2.2, "Nearby Industrial, Transportation, and Military Facilities," Section 2.2.1 of the AP1000 DCD is renumbered as Section 2.2.4.

AP1000 COL Information Item

- LNP COL 2.2-1

The applicant provided additional information in LNP COL 2.2-1 to resolve COL Information Item 2.2-1 (COL Action Item 2.2-1), which addresses information about industrial, military, and transportation facilities and routes to establish the presence and magnitude of potential external hazards.

2.2.1.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

The acceptance criteria associated with the relevant requirements of the Commission regulations for the nearby industrial, transportation, and military facilities are given in NUREG-0800, Sections 2.2.1-2.2.2.

The applicable regulatory requirements for identifying locations and routes are:

- 10 CFR 100.20(b), which requires that the nature and proximity of man related hazards (e.g., airports, dams, transportation routes, military and chemical facilities) be evaluated to establish site parameters for use in determining whether plant design can accommodate commonly occurring hazards, and whether the risk of other hazards is very low.

- 10 CFR 52.79(a)(1)(iv), as it relates to the factors to be considered in the evaluation of sites, which require the location and description of industrial, military, or transportation facilities and routes, and of 10 CFR 52.79(a)(1)(vi) as it relates to the compliance with reactor site criteria in 10 CFR Part 100.

- In addition, in accordance with 10 CFR Part 52, Appendix D, Section VIII, the applicant identified a Tier 2 departure, which does not require prior Commission approval. This departure is subject to the requirements in Section VIII, which are similar to the requirements in 10 CFR 50.59, "Changes, tests and experiments."

The related acceptance criteria from Sections 2.2.1 and 2.2.2 of NUREG-0800 are as follows:

- Data in the FSAR adequately describes the locations and distances from the plant for nearby industrial, military, and transportation facilities and that such data are in agreement with data obtained from other sources, when available.

- Descriptions of the nature and extent of activities conducted at the site and in its vicinity, including the products and materials likely to be processed, stored, used, or transported, are adequate to permit identification of the possible hazards cited in Section III of Sections 2.2.1-and 2.2.2 of NUREG-0800.

- Sufficient statistical data with respect to hazardous materials are provided to establish a basis for evaluating the potential hazards to the plant or plants considered at the site.

The regulatory requirement associated with the Tier 2 departure request is as follows:

- 10 CFR Part 52, "Licenses, certifications, and approvals for nuclear power plants," Appendix D, "Design Certification Rule for the AP1000 Design," Section VIII, "Processes for Changes and Departures," Item B.5.

2.2.1.4 *Technical Evaluation*

The NRC staff reviewed Section 2.2 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to nearby industrial, transportation, and military facilities. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

Tier 2 Departure

- STD DEP 1.1-1

The applicant's evaluation in accordance with Item B.5 of Section VIII of Appendix D to 10 CFR Part 52 determined that this departure did not require prior NRC approval. The NRC staff finds that it is reasonable that the departure does not require prior NRC approval because the numbering system proposed by the applicant does not alter the information required to be provided. A detailed evaluation of STD DEP 1.1-1 and the associated exemption can be found in Section 1.5.4 of this SER.

AP1000 COL Information Item

- LNP COL 2.2-1

The NRC staff reviewed LNP COL 2.2-1 related to information about industrial, military, and transportation facilities and routes to establish the presence and magnitude of potential external hazards included in Section 2.2 of the LNP COL FSAR. COL information item in AP1000 DCD Section 2.2.1 states:

> Combined License applicants referencing the AP1000 certified design will provide site-specific information related to the identification of potential hazards within the site vicinity, including an evaluation of potential accidents and verify that the frequency of site-specific potential hazards is consistent with the criteria outlined in Section 2.2. The site-specific information will provide a review of aircraft hazards, information on nearby transportation routes, and information on potential industrial and military hazards.

The NRC staff reviewed the LNP COL FSAR using the guidance described in Sections 2.2.1 and 2.2.2 of NUREG-0800.

This SER section identifies and provides the information that would help in evaluating potential effects on the safe operation of the nuclear facility by industrial, transportation, mining, and military installations in the LNP area. The evaluation of potential effects on the safe operation of the nuclear facility is described in SER Section 2.2.3.

Locations and Routes

The applicant identified and provided information regarding potential external hazard facilities and operations within a 5 mile radius of the LNP site.

The NRC staff confirmed that no major industrial activities are located within the 8-kilometer (km) (5-mile) radius of the LNP site (FSAR Figure 2.2.1-202)

The NRC staff verified that no active quarrying or mining facilities are located within the 8-km (5-mile) radius of the LNP site. Sixteen active mining or quarrying facilities are located within a 40-km (25-mile) radius of the LNP site (FSAR Figure 2.2.1-203).

Plum Creek Timberlands, L.P., is planning a mining operation, Titan Mines–Phase 2, within 8 km (5 mile) of the LNP site, approximately 1.6 km (1 mile) west of U.S. Highway 19 (FSAR Figure 2.2.1-203). The NRC staff verified with the Titan Mines Manager that all blasting will be done by a licensed contractor and that no explosives will be stored onsite. Only the explosives for one shoot will be brought to the site each day that a shoot is scheduled.

In addition to Orlando and Tampa, which are located beyond the 80-km (50-mile) radius, Gainesville and Ocala are two major transportation hubs for central Florida that are located within the region (FSAR Figure 2.2.1-201). Gainesville and Ocala are served by rail lines, as well as major interstates and highways that serve local and interstate traffic. These highways and interstates are described in LNP COL FSAR Section 2.2.2.5.

The NRC staff verified that no airports or private airstrips are located within the 8-km (5-mile) radius of the LNP site (FSAR Figure 2.2.1-204). J.R's private airstrip and the Crystal River Power Plant Heliport are located within a 16-km (10-mile) radius of the site.

Military Facilities

The NRC staff verified that no active military facilities are within 8 km (5 mile) of the LNP site. Florida National Guard, Company B, 3rd Battalion, 20th Special Forces Group and the 690th Military Police Company National Guard are the only significant military facilities located within an 80-km (50-mile) radius of the LNP site. Florida National Guard, Company B, 3rd Battalion, 20th Special Forces is located in Brooksville, Florida and is 67.6 km (42 mile) from the site. The 690th Military Police Company National Guard is located in Crystal River, Florida, adjacent to the Crystal River Airport, and is 24.5 km (15.2 mile) from the site.

Railroads

The NRC staff verified that no active railroads are located within the 8-km (5-mile) radius of the LNP site. Two railroad lines, an abandoned track, and an active line are located within 16 km (10 mile) of the LNP site.

Manufacturing and Storage of Hazardous Materials

The NRC staff verified that no manufacturing facilities that use or store hazardous products are located within the 8-km (5-mile) radius of the LNP site (FSAR Figure 2.2.2-201). A Tier 2 facility (the Town of Inglis water treatment plant [WTP]) is located approximately 4.8 km (3 mile) from the LNP site and stores/uses hazardous chemicals. Tier 2 facilities are those that store or manufacture hazardous materials. LNP COL FSAR Table 2.2.2-202 presents the chemicals and the quantities stored/used at the Town of Inglis WTP.

The NRC staff verified the following information. Florida Public Utilities is located on the east side of U.S. Highway 19, approximately 5.5 km (3.4 mile) south of the LNP site. This facility, located in the Town of Inglis, provides propane gas and has three tanks on site. One tank has a storage capacity of 113,563 liters (30,000 gallons) and each of the other two tanks can store 68,137 liters (18,000 gallons). No other volatile materials are located at this facility.

Pipelines

The NRC staff verified that underground natural gas pipelines are located within the 8-km (5-mile) radius of the LNP site on the north side of U.S. Highway 19 alongside the remaining rail bed from the abandoned railroad track. The pipelines run parallel to U.S. Highway 19, approximately 1769 meters (m) (5803 feet [ft.]) to the west-northwest of the LNP site. Florida Gas Transmission Company (FGT) plans to construct a 24.5-km (15.2-mile) loop, which would extend approximately 24 km (15 mi) along the eastern side of the existing pipeline. In a letter dated July 14, 2011, the applicant provided additional information related to a FGT expansion project, which placed a 36-inch pipeline into service on April 1, 2011.

The 20.3-centimeter (cm) (8-inch [in.]), 76.2-cm (30-in.), and 91.4-cm (36-in.) natural gas pipelines are owned by FGT. The 20.3-cm (8-in.) pipeline is buried to a minimum of 0.9 m (3 ft.) below ground surface (bgs), and is 2123 m (6966 ft.) west of the LNP site. The pipeline has a maximum pressure of 912 pounds per square inch (psi). The 76.2-cm (30-in.) pipeline is buried a minimum of 0.9 m (3 ft.) bgs. The pipeline has a maximum pressure of 1200 psi and is located 1769 m (5803 ft) west of the LNP site. The 91.4-cm (36-in.) pipeline is buried a minimum of 0.9 m (3 ft.) bgs. The pipeline has a maximum pressure of 1333 psi and is located 1757 m (5763 ft.) west-northwest of the LNP site. There are no plans to carry any other product in the pipeline except for natural gas. The locations of the 20.3-cm (8-in.), 76.2-cm (30-in.), and 91.4-cm (36-in.) pipelines with respect to the safety-related structures of the LNP are shown in LNP COL FSAR Figure 2.2.2-202.

Description of Waterways

The NRC staff verified that five waterways are located within the 8-km (5-mile) radius of the LNP site. The waterways include Ten Mile Creek, which connects to Cow Creek and the Gulf of Mexico, Spring Run Creek, which extends to the Gulf of Mexico, Lake Rousseau, the Cross Florida Barge Canal (CFBC), and Withlacoochee River. Lake Rousseau's main channel is 4.3 to 5.2 m (14 to 17 ft) deep, the CFBC is 3.7 m (12 ft) deep, and Withlacoochee River is 3 m (10 ft) deep.

Recreational boating within the 8-km (5-mile) radius is likely to be associated with Cow Creek, Lake Rousseau, the CFBC, and Withlacoochee River. The CFBC was renamed the Marjorie Harris Carr Cross Florida Greenway and is now used for recreational boating (see LNP COL FSAR Figure 2.1.3-204). The Inglis Mine utilizes the section of the barge canal to the west of U.S. Highway 19. The Inglis Mine has a slip on the northern side of the CFBC that is used for periodic shipments of limestone. The Inglis Mine is located outside of the 8-km (5-mile) radius of the LNP site (LNP COL FSAR Figure 2.2.1-203).

Description of Highways

The NRC staff verified that the major highway located near the LNP site leading to Gainesville and Ocala is U.S. Highway 19/98 (State Route [SR] 55). LNP COL FSAR Figure 2.2.1-201 illustrates the transportation routes in the region of the LNP site. Interstate 75 (I-75) is the closest interstate, which is located approximately 45 km (28 mile) to the east of the LNP site. At its nearest point, U.S. Highway 19/98 (SR 55) is located approximately 1974 m (6477 ft) from the center of the LNP site (FSAR Figure 2.2.2-201). The average annual daily traffic (AADT) counts at the four closest monitoring points within the 8-km (5-mile) radius of the LNP site range from 1600 (Site 340086–SR 121, 0.32 km [0.2 mile] northeast of SR 55) to 8600 (Site 340069-SR 55 at the southern city limits of Inglis) vehicles per day. This highway is mainly used for local traffic and local commodity deliveries.

Description of Railways

The NRC staff verified that two railroad lines are located within 16 km (10 mile) of the LNP site. The lines include an abandoned track with only the rail bed remaining, which is located northeast of the site and north of SR 336, and an active railroad line operated by CSX Transportation, Inc. (CSX), which is located southeast of the LNP site. The CSX line runs from the city of Crystal River northeast to the city of Dunnellon. The applicant stated that in accordance with NRC RG 1.206, further analysis of the CSX rail segment was not required since it is outside of the 8-km (5-mile) radius of the LNP site. RG 1.206, Section C.1.2.2, footnote 2, states that applicants should consider all facilities and activities within 5 miles (8.05 km) of the nuclear site. NUREG-0800, Section 2.2.1-2.2.2, item III.2, states that the staff's review should include all identified facilities and activities within 8 kilometers (5 miles) of the plant. The staff confirmed that no railroad passes within 5 miles of the LNP site. The staff finds that not performing additional analysis of the CSX rail segment is acceptable because it meets the criteria described in NUREG-0800 and the guidance of RG 1.206.

Description of Airports

The NRC staff verified that no airports are within the 8-km (5-mile) radius of the LNP site (LNP COL FSAR Figure 2.2.1-204). J.R.'s private airstrip is 10.1 km (6.3 mile) from the LNP site, and the Crystal River Power Plant Heliport is 14.5 km (9 mile) from the site. Nine public airports and 48 private airports or airstrips are located outside the 16-km (10-mile) radius, but within the 80-km (50-mile) radius of the LNP site, but these locations have limited facilities. No further analysis was performed by the applicant on the private airports or airstrips. The nine public airports and their respective distances to the LNP site are listed in LNP COL FSAR Section 2.2.2.7. LNP COL FSAR Table 2.2.2-203 provides a summary of operations data for these public airports. The table includes distance to the LNP site, daily operation traffic, runway information types of aircraft using the facility, aircraft based on the field, and flying patterns associated with each airport.

Approximately 50 aircraft are based at the Crystal River Airport (43 single-engine, 5 multi-engine, 1 helicopter, and 1 glider airplane), with approximately 100 aircraft operations per day (49 percent local general aviation [49 flights]; 49 percent transient general aviation [49 flights]; 1 percent air taxi aviation [1 flight]; and less than 1 percent military [1 flight]). Future plans for the airport include a 1524-m (5000-ft) extension of the east-west runway to be completed within the next 4 to 5 years. This improvement is designed to make aircraft landings safer and will not increase traffic. No aircraft accidents or collisions have occurred at Crystal River Airport that have resulted in fatalities or that have been considered serious accidents. Only minor landing mishaps that did not result in property damage have been reported by airport operations.

Approximately 52 aircraft are based at Marion County Dunnellon Airport (42 single-engine, 5 multi-engine, and 5 ultra lights), with approximately 41 aircraft operations per day (80 percent local general aviation [33 flights] and 20 percent transient general aviation [8 flights]). Future plans for the airport include rehabilitation of the two existing runways to accommodate slightly larger general aviation and corporate aircraft. An increase in traffic is not expected. Two accidents occurred in the past 3 years at Marion County Dunnellon Airport.

Approximately 36 aircraft are based at Williston Municipal Airport (27 single-engine, 3 multi-engine, 2 jet planes, 2 helicopters, and 2 ultra lights), with approximately 45 aircraft operations per day (30 percent local general aviation [14 flights] and 70 percent transient general aviation [31 flights]). Skydiving activities also originate from the Williston Municipal Airport. Williston Municipal Airport will be constructing new hanger storage and anticipates a 20 percent growth in operations. No aircraft accidents or collisions have occurred at Williston Municipal Airport that have resulted in fatalities or that have been considered serious accidents. Only minor landing mishaps that did not result in property damage have been reported by airport operations.

The closest large-scale public airport to the LNP site is the Ocala International Airport (LNP COL FSAR Figure 2.2.1-204). Ocala International Airport maintains 155 aircrafts used for general aviation with approximately 110,000 operations annually. No plans to expand the runways are projected for the near future at Ocala International Airport; however, within in the next 10 to 15 years, the airport plans to expand. Consistent with the guidance in NUREG-0800, Section 3.5.1.6 and RG 1.106, Section C.1.2.2.2.7, and due to Ocala's distance from the LNP site, Ocala International Airport operations would have to increase more than 500% before the applicant would have to provide an additional analysis regarding the probability of an airplane crash affecting safety related structures or systems at the LNP site.

George T. Lewis Airport, also known as the Cedar Key Airport, is located on an island 1.6 km (1 mile) west of Cedar Key and is owned by Levy County. The airport is public, does not have service staff, and has very light operations. George T. Lewis Airport has no aircraft types or operations data and has no plans to expand. The main function of this airport is to serve the resort and recreation activities at Cedar Key.

The Hernando County Airport maintains166 total aircraft with approximately 72,500 annual operations (125 single-engine, 16 twin-engine, 8 jets, 15 helicopters, and 2 ultra lights). Currently, the airport is extending one of the runways. No major accidents have been reported.

Approximately 135 aircrafts are based at the Gainesville Regional Airport, with 93,502 annual operations. Helicopters for the Gainesville Police and Alachua County Sheriff's Department are also housed at this airport, in addition to operating a flight school. Additional growth for the airport will be associated with the Eclipse 500.

LNP COL FSAR Table 2.2.2–203 describes the types of aircraft and flying patterns for aircraft-associated airports within the region. According to the Federal Aviation Administration (FAA), there are no temporary flight restrictions (TFR) within 32 km (20 mile) of the LNP site.

The applicant addressed and evaluated potential aircraft hazards following the approach and methodology outlined in NUREG-0800, Section 3.5.1.6, "Aircraft Hazards," and determined an aircraft crash into the effective plant areas of the safety-related structures on the site met the acceptance criteria. One of the factors the applicant used to assess the probability of aircraft accidents resulting in radiological consequences greater than the 10 CFR Part 100 exposure guidelines, was that there were no Federal airways within 2 miles of the LNP site.

In a letter dated March 6, 2009, the staff requested additional information (RAI) related to Federal airways within the 2 mile radius of the LNP site and requested that the applicant address the potential hazards. The applicant response to this RAI, dated April 6, 2009, noted a total of five Federal airways within the 2 mile limit of the LNP site.

The applicant submitted a supplemental response to this RAI, dated July 29, 2009. This supplement provided an analysis of the potential hazards from these airways and revised the LNP COL FSAR Sections 2.2.2.7 and 3.5.1.6. The applicant also replaced Table 2.2.1-204 and added new Table 3.5-201. The staff found the applicant's analysis showing the large and small aircraft crash probabilities, to be acceptable.

The staff reviewed this evaluation, the methodology and acceptance criteria and determined that the application is consistent with the acceptance criteria in NUREG–0800 Section 3.5.1.6.

The staff verified that the proposed markup changes in the applicant's RAI response are acceptable. This RAI is closed.

Projections of Industrial Growth

The staff verified that the LNP site is located in the southern part of Levy County immediately east of U.S. Highway 19/98 (SR 55). The site is primarily timber and currently undeveloped. The Goethe State Forest is located to the northeast, and the surrounding area is undeveloped agricultural land or sparsely populated rural residential land use. Some commercial automotive service, parts, storage, and gas stations are located within 8 km (5 mi) of the site. These facilities are primarily located along U.S. Highway 19 and County Route 40. Because Levy County is primarily rural; the majority of the industrial development within an 80-km (50-mile) radius of the LNP site is located in the urbanized areas of Marion and Citrus counties. Personal communication with the Levy County Planning Department indicates that no industrial growth is planned within an 8-km (5-mile) radius of the project site. Industrial development within a 16-km (10-mile) radius of the LNP site is primarily concentrated in Inglis along County Route 40 and U.S. Highway 19, and is limited to metal fabrication, automotive repair shops, and several mining operations. Mines within the 16-km (10-mile) radius of the LNP site include the Inglis Mine, located north of the CFBC; Holcim (US), Inc., located south of the CFBC; and Crystal River Quarry located in the community of Red Level. Gulf Rock Mine is located northwest of the LNP site and is inactive (LNP COL FSAR Figure 2.2.1-203).

The LNP site is located in the southern portion of Levy County. Citrus County is located to the south and Marion County is located to the east. LNP COL FSAR Table 2.2.2-204 lists the largest employers in Citrus, Levy, and Marion counties. The largest employers are within the utilities, education, and healthcare sectors.

2.2.1.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.2.1.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to nearby industrial, transportation, and military facilities, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the applicant has presented and substantiated information to establish an identification of potential hazards in the site vicinity. The staff has reviewed LNP COL 2.2-1, and for the reasons given above, concludes that the applicant has provided information with respect to identification of potential hazards in accordance with the requirements of 10 CFR 52.79(a)(1)(iv) and 10 CFR 52.79(a)(1)(vi). The nature and extent of activities involving potentially hazardous materials that are conducted at nearby industrial, military, and transportation facilities have been evaluated to identify any such activities that have the potential for adversely affecting plant safety-related structures. Based on an evaluation of information in the LNP COL FSAR, as well as information that the staff independently obtained, the staff has concluded that all potentially hazardous activities on site and in the vicinity of the plant have been identified. The hazards associated with these activities have been reviewed and are discussed in Sections 2.2.3, 3.5.1.5, and 3.5.1.6 of this SER.

The staff also concluded that STD DEP 1.1-1 meets the requirements for departures in 10 CFR Part 52, Appendix D, Section VIII, Item B5 and is, therefore, acceptable.

2.2.2 Descriptions

The NRC staff's review of the LNP COL FSAR Section 2.2.2, "Descriptions," is addressed in SER Section 2.2.1.

2.2.3 Evaluation of Potential Accidents

2.2.3.1 *Introduction*

The evaluation of potential accidents considers the applicant's probability analyses of potential accidents involving hazardous materials or activities on site and in the vicinity of the proposed site to confirm that appropriate data and analytical models have been used. The review covers the following specific areas: (1) hazards associated with nearby industrial activities, such as manufacturing, processing, or storage facilities, (2) hazards associated with nearby military activities, such as military bases, training areas, or aircraft flights, and (3) hazards associated with nearby transportation routes (aircraft routes, highways, railways, navigable waters, and pipelines). Each hazard review area includes consideration of the following principal types of hazards: (1) toxic vapors or gases and their potential for incapacitating nuclear plant control room operators, (2) overpressure resulting from explosions or detonations involving materials such as munitions, industrial explosives, or explosive vapor clouds resulting from the atmospheric release of gases (such as propane and natural gas or any other gas) with a potential for ignition and explosion, (3) missile effects attributable to mechanical impacts, such as aircraft impacts, explosion debris, and impacts from waterborne items such as barges, and (4) thermal effects attributable to fires.

The scope of the review also includes the evaluation of man-made site hazards that have been identified as design-basis accidents with respect to safety-related structures, systems, and components (SSCs).

2.2.3.2 *Summary of Application*

Section 2.2 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.2 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.2, the applicant provided the following:

AP1000 COL Information Item

- LNP COL 2.2-1

The applicant provided additional information in LNP COL 2.2-1 to resolve COL Information Item 2.2-1 (COL Action Item 2.2-1), which addresses the provision of information about industrial, military, and transportation facilities and routes to establish the presence and magnitude of potential external hazards, including the following accident categories: explosions, flammable vapor clouds (delayed ignition), toxic chemicals, fires, and airplane crashes.

2.2.3.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the evaluation of potential accidents are given in Section 2.2.3 of NUREG-0800.

The applicable regulatory requirements for evaluation of potential accidents are:

- 10 CFR 100.20(b), which requires that the nature and proximity of man-made related hazards (e.g., airports, dams, transportation routes, military and chemical facilities) be evaluated to establish site parameters for use in determining whether plant design can accommodate commonly occurring hazards, and whether the risk of other hazards is very low.

- 10 CFR 52.79(a)(1)(iv), as it relates to the factors to be considered in the evaluation of sites, which require the location and description of industrial, military, or transportation facilities and routes, and the requirements of 10 CFR 52.79(a)(1)(vi) as they relate to compliance with 10 CFR Part 100.

The related acceptance criteria from Section 2.2.3 of NUREG-0800 are as follows:

- Event Probability: The identification of design-basis events (DBEs) resulting from the presence of hazardous materials or activities in the vicinity of the plant or plants of specified type is acceptable if all postulated types of accidents are included for which the expected rate of occurrence of potential exposures resulting in radiological dose in excess of the 10 CFR 50.34(a)(1) limits as it relates to the requirements of 10 CFR Part 100, is estimated to exceed the NRC staff's objective of an order of magnitude of 10^{-7} per year.

- Design-Basis Events: The effects of DBEs have been adequately considered, in accordance with 10 CFR 100.20(b), if analyses of the effects of those accidents on the safety-related features of the plant or plants of specified type have been performed and

> measures have been taken (e.g., hardening, fire protection) to mitigate the consequences of such events.

In addition, the toxic gas evaluations should be consistent with appropriate sections from RG 1.78, "Evaluating the Habitability of a Nuclear Power Plant Control Room During a Postulated Hazardous Chemical Release," Revision 1(December 2001).

2.2.3.4 *Technical Evaluation*

The NRC staff reviewed Section 2.2 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the evaluation of potential accidents. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

AP1000 COL Information Item

- LNP COL 2.2-1

The NRC staff reviewed the LNP COL 2.2-1 related to information about industrial, military, and transportation facilities and routes used to establish the presence and magnitude of potential external hazards, including the following accident categories: explosions, flammable vapor clouds (delayed ignition), toxic chemicals, fires, and airplane crashes included in Section 2.2.3 of the LNP COL FSAR. COL information item in Section 2.2 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will provide site-specific information related to the identification of potential hazards within the site vicinity, including an evaluation of potential accidents and verify that the frequency of site-specific potential hazards is consistent with the criteria outlined in Section 2.2. The site-specific information will provide a review of aircraft hazards information on nearby transportation routes, and information on potential industrial and military hazards.

Explosions

The applicant considered hazards involving potential explosions that could result in blast overpressure due to detonation of explosives, chemicals, liquid fuels, and gaseous fuels for facilities and activities either onsite or within the site vicinity of the proposed units. The applicant evaluated potential explosions from nearby highways, railways, or facilities using 1 psi overpressure as a criterion for adversely effecting plant operation or preventing safe shutdown of the plant. In accordance with RG 1.91, "Evaluation of Explosions Postulated to Occur on

Transportation Routes Near Nuclear Power Plants," peak positive incident overpressures below 1 psi are considered to cause no significant damage.

The applicant determined a minimum safe standoff distance of 1658 ft. for truck transport using conservative assumptions and RG 1.91 methodology. By comparison, the distance to the closest highway is 6477 ft. from the nearest safety-related structure. The NRC staff performed independent calculations, which confirmed the applicant's results. Therefore, the NRC staff concludes the applicant's assumptions and methodology are acceptable, because they follow the guidance described in RG 1.91.

The applicant reported that, except for minor barge traffic on the CFBC, to and from the Inglis Mine (approximately 6 miles from the LNP), the local waterways are not navigable for commercial shipping and therefore, are not considered for hazard evaluations. The NRC staff finds this determination acceptable, recognizing that the CFBC may be used for the delivery of components during the construction of LNP.

In a letter dated July 14, 2011, the applicant provided additional information related to a FGT expansion project, which placed a 36-inch pipeline into service on April 1, 2011. The nearest and largest nearby natural gas pipeline runs parallel to U.S. Highway 19, approximately 5763 ft. to the west-northwest of LNP as shown on FSAR Figure 2.2.2-202. The 36-inch diameter pipeline is buried at a depth of 3 ft. with a maximum operating pressure of 1333 psi. Isolation of the line is obtained with isolation valves up to 19.4 miles apart.

In LNP COL FSAR Section 2.2.3.2.3, the applicant stated that unconfined vapor explosions of natural gas are not considered credible events. The applicant also stated that deflagration of a natural gas/air mixture is the limiting case, assuming that a mixture within the flammable limits is not present near the safety-related structures. In FSAR Section 2.2.3.2.3, a delayed flammable cloud ignition is discounted on the basis of insufficient gas concentrations at the LNP site. However, resolving this issue for an onsite hazard does not preclude ignition at a location between the pipeline and the LNP site. Therefore, the overpressure hazard from either immediate or delayed ignition of the vapor cloud is not resolved. In a letter dated March 6, 2009, the NRC staff requested clarification of the applicant's statement that unconfined vapor explosions of a natural gas/air mixture are not credible.

In a letter dated April 6, 2009, the applicant provided a revision to LNP COL FSAR Section 2.2.3.2.3 to clarify the overpressure analysis, and clarified the basis for the statement that unconfined vapor explosions of a natural gas/air mixture are not credible. In the July 14, 2011, letter related to the 36-inch pipeline, the applicant affirmed the overpressure analysis and associated technical basis described in the April 6, 2009, letter and in FSAR section 2.2.3.2.3.

The NRC staff verified the analysis and determined the clarification requested is acceptable, because it follows the guidance described in RG 1.91. This RAI is closed.

Toxic Chemicals

The applicant stated, there is no rail or major barge traffic within 8 km (5 miles) of the LNP site. The road transportation corridors within 8 km (5 miles) of the LNP site include the following routes. U.S. Highway 19/98, located 1.9 km (1.2 miles) west of the LNP site, is mainly used for local traffic and local commodity deliveries only. Four county roads are shown on Figure 2.2.2-201: County Road 40, 4.5 km (2.8 miles) south; County Road 40A, 4.8 km (3.0 miles) southwest; SR 336, 6.8 km (4.2 miles) east-northeast; and County Road 337, 7.7 km (4.8 miles) northeast of the LNP site. None of these roadways are assumed to carry regular heavy truck traffic. Due to the lack of major industries in the area, significant commodity traffic on U.S. Highway 19/98 is expected to be minimal, with the preferred route for north-south commodity flow to be via I-75, which is 45.1 km (28 miles) east of the LNP site. Therefore, there are no adverse effects to LNP likely due to the transportation of toxic materials. The NRC staff, after independently reviewing available information on the internet from local, State and Federal agencies, concluded that the applicant's determination is adequate.

The applicant stated further that stationary hazardous chemical sources within 8 km (5 miles) of the LNP site are limited to the Inglis WTP located 4.8 km (3 mi) from the LNP site. As listed in Table 2.2.2-202, the quantities stored at the plant are small and are not significant sources of airborne contamination even in the event of an accidental failure of the storage containers. Therefore, there are no offsite sources of toxic chemicals within 8 km (5 miles) of the LNP site that could pose a threat to LNP.

In response to RAI 2.2.1-2.2.2-1 pertaining to the onsite storage of chemicals, the applicant stated that the chemicals stored on site are bounded by the standard chemicals identified in DCD Table 6.4-1. These chemicals were assessed by Westinghouse as part of the main control room habitability hazard analysis. The Westinghouse analysis found the chemicals listed in AP1000 DCD Table 6.4-1 not to present a hazard to the control room operators or to safety-related SSCs.

The applicant identified no site specific onsite toxic chemicals other than the standard onsite toxic chemicals identified in LNP COL FSAR Table 6.4-201. The NRC staff finds the chemicals listed in LNP COL FSAR Table 6.4-201 to be acceptable because they follow the guidance described in RG 1.78.

Fires

Fires originating from accidents at any facilities or transportation routes identified above do not have the potential to affect the safe operation of LNP because the distances between potential accident locations and LNP are greater than 1.6 km (1 mile). The closest potential source of a significant fire is the 91.4-cm (36-in.) natural gas line at 1757 m (5763 ft.) from the LNP site. An evaluation of the heat flux from a prolonged fire at the gas line results in a calculated heat flux less than the maximum solar heat flux on the surface of the earth (approximately 300 British thermal units per hour per square foot) at about 883.9 m (2900 ft) from the pipeline. In addition, the LNP main control room heating, ventilation, and air conditioning (HVAC) system continuously monitors the outside air using smoke monitors located at the outside air intake plenum and monitors the return air for smoke upstream of the supply air handling units (DCD Section 9.4.1.2.3.1). If a high concentration of smoke is detected in the outside air intake, an alarm is initiated in the main control room and the main control room/technical support center HVAC subsystem is manually realigned to the recirculation mode by closing the outside air and toilet exhaust duct isolation valves. Therefore, any potential heavy smoke problems at the main control room air intakes would not affect the LNP operators. The NRC staff reviewed the above information and concluded that the applicant's determination is acceptable because it follows the guidance described in RG 1.78 and RG 1.106.

Collision with the Intake Structure

This section is not applicable, as the LNP intake structure is not located on a navigable waterway with commercial traffic.

Liquid Spills

There is no safety-related equipment located at the intake structure. The CFBC is now used for recreational boating. In addition, the Inglis Mine utilizes a section of the CFBC to the west of U.S. Highway 19 periodically for minor barge shipments of limestone. Neither category of water traffic is considered likely to possess or transport liquids that may be corrosive, cryogenic, or coagulant. Accidental release of minor quantities of oil could be associated with marine engine operation but would be effectively diluted by the water in the CFBC and Gulf of Mexico.

Therefore, in the unlikely event of an accidental spill of oil or liquids that may be corrosive, cryogenic, or coagulant in nature, the CFBC would provide ample dilution before any such liquids reach the CWS. Even if the operation of the CWS were adversely affected by an accidental spill, there would be no impact on the ability of the plant to safely shutdown since the passive core cooling system would not be affected by degradation of the CWS.

The NRC staff reviewed this information and finds it acceptable because the CWS of the AP1000 design has no safety related function.

2.2.3.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.2.3.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to evaluation of potential accidents, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As discussed above, the applicant has presented and substantiated information to identify potential hazards in the site vicinity. The staff has reviewed the information provided and concludes that the applicant has provided sufficient information with respect to the identification of potential hazards in accordance with the requirements of 10 CFR 52.79(a)(1)(iv) and 10 CFR 52.79(a)(1)(vi). The nature and extent of activities involving potentially hazardous materials that are conducted at nearby industrial, military, and transportation facilities have been evaluated to identify any such activities that have the potential for adversely affecting plant safety-related structures. Based on an evaluation of information in the LNP COL FSAR as well as information that the staff independently evaluated, the staff has concluded that potentially hazardous activities on site and in the vicinity of the LNP site have been identified. This addresses and resolves COL Information Item 2.2-1. In conclusion, the applicant has provided sufficient information to satisfy the requirements of 10 CFR Parts 50, 52, and 100.

2.3 Meteorology

To ensure that a nuclear power plant or plants can be designed, constructed, and operated on an applicant's proposed site in compliance with the NRC regulations, the NRC staff evaluates regional and local climatological information, including climate extremes and severe weather occurrences that may affect the design and siting of a nuclear plant. The staff reviews information on the atmospheric dispersion characteristics of a nuclear power plant site to determine whether the radioactive effluents from postulated accidental releases, as well as routine operational releases, comply with NRC regulations. The staff has prepared Sections 2.3.1 through 2.3.5 of this safety evaluation report (SER) in accordance with the review procedures described in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)," using information presented in Section 2.3 of the LNP COL FSAR (which references Revision 19 to the AP1000 DCD), responses to staff requests for additional information (RAIs), and generally available reference materials (as cited in applicable sections of NUREG-0800).

2.3.1 Regional Climatology

2.3.1.1 *Introduction*

Section 2.3.1, "Regional Climatology," of the LNP COL FSAR addresses averages and extremes of climatic conditions and regional meteorological phenomena that could affect the safe design and siting of the plant, including information describing the general climate of the region, seasonal and annual frequencies of severe weather phenomena, and other meteorological conditions to be used for design- and operating-basis considerations.

2.3.1.2 *Summary of Application*

Section 2.3 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.3 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.3, the applicant provided the following:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.3-1

The applicant provided additional information in LNP COL 2.3-1 to address COL Information Item 2.3-1 (COL Action Item 2.3.1-1). LNP COL 2.3-1 addresses site-specific information related to regional climatology.

In addition, this LNP COL FSAR section addresses Interface Item 2.4 related to extreme meteorological conditions for the design of systems and components exposed to the environment, Interface Item 2.5 related to tornado and operating basis wind loadings, Interface Item 2.7 related to snow, ice and rain loads, and Interface Item 2.8 related to ambient air temperatures.

2.3.1.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793, "Final Safety Evaluation Report Related to Certification of the AP1000 Standard Design," and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for regional climatology are given in Section 2.3.1 of NUREG-0800.

The applicable regulatory requirements for identifying regional meteorology are:

- Title 10 of the *Code of Federal Regulations* (10 CFR) 52.79(a)(1)(iii), as it relates to identifying the more severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated.

- 10 CFR 100.20(c)(2), and 10 CFR 100.21(d), with respect to the consideration given to the regional meteorological characteristics of the site.

The climatological and meteorological information assembled in compliance with the above regulatory requirements are necessary to determine a proposed facility's compliance with the following requirements in Appendix A of 10 CFR Part 50:

- GDC 2, Design Bases for Protection Against Natural Phenomena, which requires that structures, systems and components important to safety be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions.

- GDC 4, Environmental and Dynamic Effects Design Bases, which requires that SSCs important to safety be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, included loss-of-coolant accidents.

The related acceptance criteria from Section 2.3.1 of NUREG-0800 are as follows:

- The description of the general climate of the region should be based on standard climatic summaries compiled by the National Oceanic and Atmospheric Administration (NOAA).
- Data on severe weather phenomena should be based on standard meteorological records from nearby representative National Weather Service (NWS), military, or other stations recognized as standard installations that have long periods of data on record.
- The tornado parameters should be consistent with Regulatory Guide (RG) 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," Revision 1. Alternatively, an applicant may specify any tornado parameters that are appropriately justified, provided that a technical evaluation of site-specific data is conducted.
- The basic (straight-line) 100-year return period 3-second gust wind speed should be based on appropriate standards, with suitable corrections for local conditions.
- Consistent with RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," Revision 2, the ultimate heat sink (UHS) meteorological data that would result in the maximum evaporation and drift loss of water and minimum water cooling should be based on long-period regional records that represent site conditions. (Not applicable to a passive containment system design that does not utilize a cooling tower or cooling pond).
- The weight of the 100-year return period snowpack should be based on data recorded at nearby representative climatic stations or obtained from appropriate standards with suitable corrections for local conditions. The weight of the 48-hour probably maximum winter precipitation (PMWP) should be determined in accordance with reports published by NOAA's Hydrometeorological Design Studies Center.
- Ambient temperature and humidity statistics should be derived from data recorded at nearby representative climatic stations or obtained from appropriate standards with suitable corrections for local conditions.
- High air pollution potential information should be based on United States Environmental Protection Agency (EPA) studies.
- All other meteorological and air quality conditions identified by the applicant as design and operating bases should be documented and substantiated.

The information should be consistent with acceptable practices, data from NOAA, industry standards, and NRC regulatory guides.

Interim staff guidance (ISG) document DC/COL-ISG-7, "Interim Staff Guidance on Assessment of Normal and Extreme Winter Precipitation Loads on the Roofs of Seismic Category I

Structures," was issued subsequent to the publication of NUREG-0800, Section 2.3.1. The ISG clarifies the staff's position that the applicant should identify winter precipitation events as site characteristics and site parameters for determining normal and extreme winter precipitation loads on the roofs of seismic Category I structures.

2.3.1.4 *Technical Evaluation*

The NRC staff reviewed Section 2.3.1 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to regional climatology. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

AP1000 COL Information Item

- LNP COL 2.3-1

The NRC staff reviewed LNP COL 2.3-1 related to the provision of regional climatology included in Section 2.3.1 of the LNP COL FSAR. The COL information item 2.3-1 in Section 2.3.6.1 of the AP1000 DCD, states:

> Combined License applicants referencing the AP1000 certified design will address site-specific information related to regional climatology.

Evaluation of the information provided in LNP COL 2.3-1 is discussed below.

2.3.1.4.1 General Climate

The applicant's description of the general climate of the proposed LNP site is based on references, which include the National Climatic Data Center (NCDC) Local Climatic Data (LCD) Annual Summaries for Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida. Airflow, temperature and humidity, and precipitation patterns for these five locations were presented in LNP COL FSAR Table 2.3.1-202. The applicant identified the LNP site as being located in Florida's North Central state climate division as specified by the NCDC.

The NRC staff compared the applicant's general climate description to a similar NCDC narrative description of the climate of Florida (NCDC, Climates of the States #60)[2] and has confirmed its accuracy and completeness; thus, the staff accepts the applicant's description of the general climate.

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[2] http://cdo.ncdc.noaa.gov/climatenormals/clim60/states/Clim_FL_01.pdf Accessed 11/17/2008

2.3.1.4.2 Regional Meteorological Conditions for Design and Operating Basis

2.3.1.4.2.1 Thunderstorms, Hail, and Lightning

The following discussion on thunderstorms, hail, and lightning is intended to provide a general understanding of the severe weather phenomena in the site region but does not result in the generation of site characteristics for use as design or operating bases.

The applicant stated that thunderstorms have been observed on an average of 67.5 to 81.3 days per year. Thunderstorms have occurred most frequently during the months of June, July, and August. Consistent with NUREG-0800, Section 2.3.1, the applicant compiled this information from the 2006 LCDs for Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida from the NCDC.

Using both 2006 and 2007 LCDs for Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa, Florida from the NCDC, the staff confirmed that thunderstorms have been observed on an average of 67.5 to 81.3 days per year. The staff agrees with the applicant that thunderstorms have occurred most frequently in the months of June, July, and August at the five observation locations.

The applicant stated that 45 hail events were reported in Levy County from January 1, 1950 through November 30, 2008. Hail stone diameters greater than 0.75 inches were recorded. Consistent with the guidance provided in NUREG-0800, Section 2.3.1, the applicant compiled this information from the NCDC. The applicant noted that the number of reported hail events has increased significantly over time, primarily as a result of increased reporting efficiency and confirmation skill. This increase in hail reports is also likely due to the increased number of targets because of urbanization. This is because there are more targets damaged by hail in urban areas than in a rural area. Using the same database, the staff was able to confirm the applicant's value of 45 hail events for Levy County, Florida during the same time frame.

The applicant stated that there are 12.52 flashes to earth per year per square kilometer on average, based on the data from Gainesville, Jacksonville, Orlando, Tallahassee, and Tampa. The staff independently evaluated this estimate based on LCDs from the same weather reporting stations from the NCDC and a method attributed to the Electric Power Research Institute (8.1 – 9.7 flashes to earth per square kilometer), a 10-year flash density map from Vaisala[3] (8 – 10 flashes to earth per square kilometer), and a 1999 paper by G. Huffines and R.E. Orville, titled "Lightning Ground Flash Density and Thunderstorm Duration in the Continental United States: 1989-96" (> 11 flashes to earth per square kilometer). Thus, the staff concludes that the applicant has provided a reasonable estimate of the frequency of lightning flashes.

Based on a mean frequency of 12.52 flashes to earth per year per square kilometer and an exclusion area for the proposed Units 1 and 2 of 5.64 square-kilometers, the applicant predicted

[3] http://www.lightningsafety.noaa.gov/stats/08_Vaisala_NLDN_Poster.pdf accessed 9/27/2010

that 70.6 lightning flashes per year can be expected within the exclusion area of the two proposed units. Using the methodology provided in Annex L of the National Fire Protection Association (NFPA), Standard for the Installation of Lightning Protection Systems, 2008 Edition, the staff has confirmed the applicant's calculation and finds it to be a reasonable estimate.

Consistent with the guidance provided in NUREG-0800, Section 2.3.1, the applicant has provided the necessary information regarding thunderstorms, hail, and lightning. As previously discussed, the staff has independently confirmed the descriptions provided by the applicant and accepts them as correct and adequate.

2.3.1.4.2.2 Tornadoes and Severe Winds

The applicant used a 57.25-year period of tornado reports (01/01/1950 through 03/31/2007) from the NCDC to determine the number of reported tornadoes in the vicinity of the proposed LNP COL site. During this period there have been 2911 total tornadoes (50.8 tornadoes per year) in Florida and 336 reported tornadoes in the 10 counties surrounding the proposed LNP site. The 10 surrounding counties include Levy, Dixie, Gilchrist, Alachua, Marion, Lake, Sumter, Citrus, Hernando, and Pasco. Using the same tornado database, the staff independently confirmed the tornado statistics, as presented in LNP COL FSAR Tables 2.3.1-203 through 2.3.1-205, as correct.

Following the methodology presented in WASH-1300, "Technical Basis for Interim Regional Tornado Criteria," issued May 1974, and the past tornado reports in the 10 counties surrounding the proposed LNP site, the applicant used the following formula to calculate the probability that a tornado will strike a particular location during any one year period:

$$P_s = \bar{n}\left(\frac{a}{A}\right)$$

where:

P_s = mean tornado strike probability per year

$\bar{n}$ = average number of tornadoes per year in the area being considered

a = average individual tornado area

A = total area being considered

The applicant calculated the probability of a tornado strike in the vicinity of the proposed LNP site of 4.39×10^{-4} per year, or, put differently, a recurrence interval of once every 2280 years. The staff verified the applicant's probabilistic calculation, using the same tornado database, "U.S. Storm Event Database, Tornadoes," from the NCDC. It should be noted that the applicant used a 1-degree square to determine the total area being considered for the tornado strike probability. This method does not follow the methodology presented in WASH-1300, which defines *A* as the "total area in which the tornado frequency has been determined." However, the total area of the 10-counties used for the tornado analysis is roughly twice that of the 1-degree square box. The applicant's method results in a higher, and consequently more conservative, estimation of the tornado strike probability due to the use of a smaller area in the denominator of the above equation. The staff also compared the tornado strike probability against the 2-degree box value in Appendix A to NUREG/CR-4461, "Tornado Climatology of the Contiguous United States," Revision 2. The staff found that the applicant has presented a conservative estimate and accepts the tornado strike probability as presented.

The applicant chose tornado site characteristics based on Draft RG 1143 (DG-1143), "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants." This draft RG provides design-basis tornado characteristics for three tornado intensity regions throughout the United States, each with a 10^{-7} per year probability of occurrence. The proposed COL site is located in tornado intensity Region II; however, the applicant has chosen to include the maximum tornado wind speed intended for tornado intensity Region I. This is a conservative assumption and is, therefore, acceptable to the staff. The applicant proposed the following tornado site characteristics, which are listed in LNP COL FSAR Table 2.0-201:

Maximum Wind Speed	300 miles per hour

Because the applicant has correctly identified those design-basis tornado site characteristics presented in DG-1143, the staff concludes that the applicant has chosen acceptable tornado site characteristics. DG-1143 was the draft Revision 1 version to RG 1.76 and is acceptable to the staff because the design-basis tornado characteristics presented are more conservative than those presented in RG 1.76, Revision 1. This is because RG 1.76, Revision 1 relies on the Enhanced-Fujita (EF) scale to relate the degree of damage from a tornado to the tornado maximum wind speed. The EF scale effectively lowered the maximum wind speed associated with tornados, thus making RG 1.76, Revision 1 values less than the values in DG-1143. The applicant stated that the latest NRC position on design basis tornadoes is based on the information in NUREG/CR-4461 Revision 1. The staff notes that the current position of the NRC on design basis tornadoes is based on NUREG/CR-4461, Revision 2, which was published in February 2007. The applicant's tornado wind speed site characteristic value bounds the value provided in NUREG/CR-4461, Revision 2 and RG 1.76, Revision 1, and is therefore acceptable to the staff.

Section 3.3.1 of the AP1000 DCD states that the operating basis wind speed site parameter value of 145 miles per hour (mph) (3-second gust) is based on an annual probability of occurrence of 0.02 (i.e., 50-year return period). Higher winds with an annual probability of occurrence of 0.01 (i.e., 100-year return period) were used in the design of seismic Category I structures by using an importance factor of 1.15. This is equivalent to designing the seismic Category I structures to a wind speed of 155 mph by using a 1.07 scaling factor from Table C6-7 of American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) 7-05, "Minimum Design Loads for Buildings and Other Structures," to convert a 50-year return period gust wind speed to a 100-year return period gust wind speed.

In an August 10, 2009, letter to the NRC, the applicant voluntarily submitted a supplemental response to RAI 2.3.1-8. In this letter, the applicant updated previous estimates of the LNP site characteristic basic wind speeds. The supplemental response to RAI 2.3.1-8 estimates that the LNP site characteristic basic wind speeds for the 50-year and 100-year return periods are 120 mph and 128 mph, respectively. The applicant followed the guidance provided in NUREG-0800, Section 2.3.1 by determining these site characteristic values using Figure 6.1 from ASCE/SEI 7-05. The staff independently verified that the applicant has followed an acceptable methodology and therefore accepts the applicant's values as correct. In RAI 2.3.1-17, the staff requested clarification on four points related to the tornado and severe wind speeds described in LNP COL FSAR Section 2.3.1.2.2. The applicant provided a clarification of each point in their RAI response and made a commitment to change and clarify the wording in the LNP COL FSAR. The staff reviewed the changes proposed in the RAI response and finds them to be acceptable. Therefore, the staff considers RAI 2.3.1-17 to be resolved. The commitment to update the FSAR with these clarifications is being tracked as **Confirmatory Item 2.3.1-1**.

Resolution of Confirmatory Item 2.3.1-1

Confirmatory Item 2.3.1-1 is an applicant commitment to update section 2.3.1 of its FSAR. The staff verified that LNP COL FSAR Section 2.3.1 was appropriately updated. As a result, Confirmatory Item 2.3.1-1 is now closed.

In RAI 2.3.1-18, the staff asked the applicant to explain the discrepancy between the 100-year return period site characteristic basic wind speed of 128 mph and the identification of a 100-year return basic wind speed of 139 mph. Using the Engineering Weather Data (EWD) compact disc, published by NOAA, the applicant updated LNP COL FSAR Section 2.3.1.2.2 to state that the maximum published 3-second gust wind speed for the region, based on severe wind events reported at the surrounding stations, is 130 mph. The applicant assumed this value represented a 50-year return period 3-second gust and converted it to a 100-year return period 3-second gust value of 139 mph using the 1.07 scaling factor from ASCE/SEI 7-05. The staff has found that the 50-year recurrence, 3-second gust basic wind speed reported on the EWD CD is based on data from ASCE 7-95, "Minimum Design Loads for Buildings and Other Structures." The 50-year recurrence basic wind speeds were updated three years later in ASCE 7-98, "Minimum Design Loads for Buildings and Other Structures," and were subsequently lowered to the basic wind speeds that are found in ASCE 7-05. The basic wind speeds presented in ASCE 7-05 were updated "based on a new and more complete analysis of hurricane wind speeds." A complete discussion on the reasons for this change can be found in Section C6.5.4, "Basic Wind Speed," of ASCE 7-05. The staff considers the 100-year return period site characteristic basic wind speed of 128 mph to be appropriate for the LNP site because it is based on the more recent analysis of hurricane winds presented in ASCE 7-05. The applicant provided clarifying language in their RAI response and made a commitment to change and clarify the wording in the FSAR. The staff reviewed the changes proposed and based on the above discussion, finds them to be acceptable. Therefore, the staff considers RAI 2.3.1-18 to be resolved. The commitment to update the FSAR with these clarifications is being tracked as **Confirmatory Item 2.3.1-2**.

Resolution of Confirmatory Item 2.3.1-2

Confirmatory Item 2.3.1-2 is an applicant commitment to update section 2.3.1 of its FSAR. The staff verified that LNP COL FSAR Section 2.3.1 was appropriately updated. As a result, Confirmatory Item 2.3.1-2 is now closed.

In RAI 2.3.1-20, the staff asked the applicant to describe how the Levy County COLA satisfies the Combined License Information requirement of AP1000 DCD Section 3.5.4 in consideration of RG 1.221, "Design-Basis Hurricane and Hurricane Missiles for Nuclear Power Plants." The applicant responded by committing to update LNP COL FSAR Subsection 3.3.2.1 and by adding new Subsection 3.5.2 and Table 3.5-202. These modifications to the FSAR, using the figures and tables in RG 1.221, include the hurricane generated missile velocities based on a maximum hurricane wind speed of 195 mph at the LNP site. The staff reviewed the changes proposed and finds them to be acceptable. Therefore, the staff considers RAI 2.3.1-20 to be resolved. The staff's evaluation of the wind loading and structural engineering aspects of RAI 2.3.1-20 is in Section 3.3.2.4 of this SER.

2.3.1.4.2.3 Heavy Snow and Severe Glaze Storms

The applicant stated that trace amounts of snowfall do occur in Florida, but measurable snowfalls are typically less than a quarter of an inch and are extremely rare. The record snowfall for the region was at Jacksonville, Florida, which received 1.5 inches of snow in February of 1958. The NRC staff issued DC/COL-ISG-007, which clarifies the NRC staff's position on identifying winter precipitation events as site characteristics and site parameters for determining normal and extreme winter precipitation loads on the roofs of seismic Category I structures. The ISG revises the previously issued NRC staff guidance as discussed in NUREG-0800, Section 2.3.1.

The ISG states that normal and extreme winter precipitation events should be identified in NUREG-0800, Section 2.3.1 as COL site characteristics for use in NUREG-0800, Section 3.8.4 in determining the normal and extreme winter precipitation loads on the roofs of seismic Category I structures. The normal winter precipitation roof load is a function of the normal winter precipitation event; whereas, the extreme winter precipitation roof loads are based on the weight of the antecedent snowpack resulting from the normal winter precipitation event plus the larger resultant weight from either: (1) the extreme frozen winter precipitation event; or (2) the extreme liquid winter precipitation event. The extreme frozen winter precipitation event is assumed to accumulate on the roof on top of the antecedent normal winter precipitation event; whereas, the extreme liquid winter precipitation event may or may not accumulate on the roof, depending on the geometry of the roof and the type of drainage provided. The ISG further states:

- The normal winter precipitation event should be the highest ground-level weight (in pounds per square foot (lb/ft^2)) among: (1) the 100-year return period snowpack; (2) the historical maximum snowpack; (3) the 100-year return period two-day snowfall event; or (4) the historical maximum two-day snowfall event in the site region.

- The extreme frozen winter precipitation event should be the higher ground-level weight (in lb/ft^2) between: (1) the 100-year return period two-day snowfall event; and (2) the historical maximum two-day snowfall event in the site region.

- The extreme liquid winter precipitation event is defined as the theoretically greatest depth of precipitation (in inches of water) for a 48-hour period that is physically possible over a 25.9-square-kilometer (km) (10-square-mile (mi)) area at a particular geographical location during those months with the historically highest snowpack.

The staff evaluated the normal winter precipitation event and the extreme frozen and liquid winter precipitation events in accordance with the ISG. Due to the location of the proposed units along the Gulf Coast, large snow and ice events are rare. The normal and extreme winter precipitation loads for the LNP COL were determined to be significantly less than the AP1000 DCD site parameter value of 75 lb/ft^2. The staff agrees with the applicant that the normal and extreme winter precipitation roof loads are not significant; therefore, the staff accepts the applicant's discussion as correct and adequate.

2.3.1.4.2.4 Hurricanes

The applicant discussed a history of hurricanes impacting both the Atlantic and Gulf of Mexico coastlines of Florida between 1899 and 2007. The applicant stated that Florida has been impacted by 150 hurricanes and tropical storms. Of the 150 storms, 85 were tropical storms, 19 were Category 1, 19 were Category 2, 19 were Category 3, 6 were Category 4, and 2 were Category 5 hurricanes. The applicant also stated that according to the NOAA Coastal Services Center (CSC), 45 hurricanes rated Category 1-5 have passed within 100 nautical mi of the LNP site. The applicant stated that the annual frequency of hurricanes is estimated to be 0.13 and 0.29 storms per year within 50- and 100-nautical mi of the LNP site, respectively.

The staff evaluated data from the NOAA CSC for hurricanes making landfall in or passing near Levy County, Florida between 1851 and 2008. The staff found that during this time period there were a total of 28 Category 1, 15 Category 2, 9 Category 3, and 2 Category 4 storms that passed within 100 nautical mi of Levy County. The staff recognizes that there are differences in the number of storms reported in the area between the staff and the applicant. However, the staff finds these differences to be small and does not consider them to have an impact on the safety analysis. Therefore, the staff accepts the applicant's descriptions of the number of hurricanes in the vicinity of Levy County, Florida.

The staff agrees with the applicant that the largest threat to the LNP site from hurricanes will be high winds, heavy precipitation, and potential flooding due to storm surges.

2.3.1.4.2.5 Normal Operating Heat Sink Design Parameters

Many plants use a cooling tower as an UHS to dissipate residual heat after an accident. Instead of using a cooling tower to release heat to the atmosphere, the AP1000 design uses a passive containment cooling system (PCS) to provide the safety-related UHS. The PCS is designed to withstand the maximum safety dry-bulb and coincident wet-bulb air temperature site parameters specified in the AP1000 DCD Tier 1 Table 5.0-1 and AP1000 DCD Tier 2 Table 2-1. Therefore, the applicant need not identify meteorological characteristics for evaluating the design of an UHS cooling tower. The applicant states in LNP COL FSAR Section 2.3.1.2.5 that the AP1000 reactor does not rely on site service water as a safety grade UHS.

A summary of statistically significant dry- and wet-bulb temperatures that were used by the applicant to determine the LNP site characteristic temperatures, as obtained from Jacksonville, Tallahassee, and Tampa, Florida, were provided in LNP COL FSAR Tables 2.3.1-207 and 2.3.1-210. These temperatures were based on the 30-year (1961-1990) Solar and Meteorological Surface Observation Network (SAMSON) database and the 23-year (1973-1996) EWD database from NOAA. The staff has performed an independent confirmatory analysis of the data provided in LNP COL FSAR Tables 2.3.1-207 and 2.3.1-210 and accepts them as correct and adequate.

The staff evaluated the applicant's design-basis temperatures primarily based on Tampa and Tallahassee, Florida hourly temperature data from 1948 to 2008 and 1943 to 2008, respectively. The Tampa, Florida observation station is located 78 mi to the south of the LNP site. This site is considered appropriate for comparison due to its close proximity to the Gulf of Mexico. The staff also compared the LNP site to the Tallahassee, Florida reporting station, which is located 138 mi northwest (NW) of the LNP site. This station was included because of its close proximity to the Gulf of Mexico. Using additional stations, as the applicant has done, such as Jacksonville, Florida, is conservative because it could only potentially result in more extreme temperatures.

Because the stations are located at approximately the same elevation as the LNP site, the staff expects that the temperature and humidity data recorded at Tampa and Tallahassee should be similar to the LNP site conditions. In order to confirm this hypothesis, the staff generated 2007 and 2008 dry-bulb statistics from the NCDC online database and compared them with similar statistics generated from the applicant's 2007 and 2008 onsite meteorological database. The results of this comparison appear below in Table 2.3.1-1:

Table 2.3.1-1. Dry-Bulb Statistics for Tampa, Tallahassee, and LNP

DRY-BULB STATISTIC	2007			2008		
	Tampa	Tallahassee	LNP	Tampa	Tallahassee	LNP
Maximum	36.0 °C	38.0 °C	34.6 °C	36.0 °C	36.0 °C	33.9 °C
1 percent Exceedance	33.0 °C	35.0 °C	32.8 °C	33.0 °C	34.0 °C	31.2 °C
Median	23.3 °C	22.0 °C	22.1 °C	24.0 °C	21.0 °C	21.4 °C
99 percent Exceedance	4.4 °C	-2.0 °C	2.0 °C	7.0 °C	-1.1 °C	-0.2 °C
Minimum	-2.0 °C	-7.2 °C	-3.9 °C	1.0 °C	-7.0 °C	-5.9 °C

The staff also compiled and compared the Tampa and Tallahassee dew point statistics with the onsite dew point data provided by the applicant (Table 2.3.1-2).

Table 2.3.1-2. Dew Point Statistics for Tampa, Tallahassee, and LNP

DEW POINT STATISTIC	2007			2008		
	Tampa	Tallahassee	LNP	Tampa	Tallahassee	LNP
Maximum	26.0 °C	27.0 °C	25.7 °C	26.0 °C	26.0 °C	25.3 °C
1 percent Exceedance	24.4 °C	24.0 °C	24.5 °C	24.0 °C	24.0 °C	24.2 °C
Median	17.2 °C	14.0 °C	17.1 °C	17.8 °C	16.0 °C	17.0 °C

This comparison shows that the Tampa and Tallahassee dry-bulb and dew point (humidity) data are generally representative of the LNP data.

Details regarding the description, design basis, and operation of the AP1000 PCS are provided in Tier 2 Section 6.2.2 of the AP1000 DCD. AP1000 DCD Section 6.2.2.1 states that the PCS is designed to withstand the effects of natural phenomena such as ambient temperature extremes. AP1000 DCD, Tier 2, Section 6.2.2.3, further states that the containment pressure analyses are based on an ambient temperature of 115 ° Fahrenheit (F) dry-bulb and 86.1 °F coincident wet-bulb. These are the maximum safety air temperature site parameter values listed in AP1000 DCD Tier 1 Table 5.0-1 and AP1000 DCD Tier 2 Table 2-1. As discussed in Section 2.3.1.4.2.7 of this SER, the applicant's site characteristic temperatures presented in LNP COL FSAR Table 2.0-201 are bounded by the AP1000 DCD site parameters.

2.3.1.4.2.6 Inversions and High Air Pollution Potential

The following discussion on inversions and high air pollution potential is intended to provide a general understanding of the phenomena in the site region but does not result in the generation of site characteristics for use as design or operating basis. NUREG-0800 states that the site's air quality should be described in detail, including identification of the site's AQCR and its attainment designation with respect to state and national ambient air quality standards.

The applicant stated that the LNP site is located in the North Central state climate division of the NCDC. The staff has confirmed that the EPA has designated that Levy County, Florida is in attainment for all criteria pollutants.

The applicant used mixing height data from Tampa, Florida to characterize the potential for inversions at the LNP site. Although Tampa, Florida is 78 mi to the south of the site, it is the closest available station with this type of data. LNP COL FSAR Table 2.3.1-208 listed the expected seasonal frequencies of inversions below 152 meters (m) (500 feet (ft.)) and LNP COL FSAR Table 2.3.1-209 listed the mean monthly mixing depths. The inversion frequency in Tampa, Florida averaged 28 percent in the summer season and 37 percent in the winter season. The lowest mean monthly mixing height occurs in January (730 m) and the greatest mean mixing depth occurs in May (1410 m). Using references[4,5] consistent with NUREG-0800, Section 2.3.1, the staff has verified that the information provided by the applicant is correct and adequate.

2.3.1.4.2.7 Ambient Air Temperatures

Along with the normal operating heat sink design temperatures presented in LNP COL FSAR Section 2.3.1.2.5 and reviewed in Section 2.3.1.4.2.5 of this SER, the applicant provided additional dry- and wet-bulb temperatures in LNP COL FSAR Section 2.3.1.2.7, which are summarized in LNP COL FSAR Table 2.3.1-10. The applicant based these additional ambient air temperature statistics on the SAMSON database, as previously discussed in SER Section 2.3.1.4.2.5, and NOAA EWD. Both of these sources are consistent with NUREG-0800, Section 2.3.1, and are acceptable to the staff. The staff relied primarily on Tampa, Florida hourly data during the period of 1938 through 2008 to review the applicant's temperatures. The results of this independent review are consistent with those presented by the applicant. Thus, the staff accepts the applicant's additional ambient temperatures as correct and adequate.

Comparison with AP1000 Site Parameters for Ambient Air Temperature

AP1000 DCD site parameters for ambient air temperature are defined as follows:

- Maximum Safety Dry Bulb Temperature and Coincident Wet-Bulb Temperature: These site parameter values represent a maximum dry-bulb temperature that exists for 2 hours

[4] Holzworth, George C., "Mixing Heights, Wind Speeds, and Potential for Urban Air Pollution Throughout the Contiguous United States," AP-101, Office of Air Programs, EPA, January 1972.

[5] J. X. L. Wang and J. K. Angell, "Air Stagnation Climatology for the United States (1948-1998)," NOAA Air Resources Laboratory Atlas No. 1, Air Resources Laboratory, Environmental Research Laboratories, Office of Oceanic and Atmospheric Research, Silver Spring, MD, April 1999.

or more, combined with the maximum wet-bulb temperature that exists in that population of dry-bulb temperatures.

- Minimum Safety Dry Bulb Temperature: This site parameter value represents a minimum dry-bulb temperature that exists within a set of hourly data for duration of 2 hours or more.

- Maximum Safety Noncoincident Wet-Bulb Temperature: This site parameter value represents a maximum wet-bulb temperature that exists within a set of hourly data for duration of 2 hours or more.

- Maximum Normal Dry-Bulb Temperature and Coincident Wet-Bulb Temperature: The maximum normal value is the 1-percent seasonal exceedance temperature. The maximum temperature is for the months of June through September in the northern hemisphere. The 1-percent seasonal exceedance is approximately equivalent to the annual 0.4-percent exceedance.

- Minimum Normal Dry-Bulb Temperature: The minimum normal value is the 99-percent seasonal exceedance temperature. The minimum temperature is for the months of December, January, and February in the northern hemisphere. The 99-percent seasonal exceedance is approximately equivalent to the annual 99.6-percent exceedance.

- Maximum Normal Noncoincident Wet-Bulb Temperature: The maximum normal value is the 1-percent seasonal exceedance temperature. The maximum temperature is for the months of June through September in the northern hemisphere. The 1-percent seasonal exceedance is approximately equivalent to the annual 0.4-percent exceedance.

The applicant's safety temperature site characteristic values are based on conservative 100-year estimates. The ambient air temperatures used for comparison against the AP1000 site parameters are presented in LNP COL FSAR Table 2.0-201.

Using a combination of NCDC hourly data from Jacksonville (1931-2008), Tallahassee (1943-2008), and Tampa (1938-2008), Florida, and climate data from the American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE), the staff was able to verify that the applicant's site characteristic temperatures presented in LNP COL FSAR Table 2.0-201 are adequate and bounded by the AP1000 DCD site parameters.

In RAI 2.3.1-19, the staff requested that the applicant update the LNP COL FSAR to change the normal ambient design site characteristic temperatures to reflect the 0.4-percent annual exceedance temperatures, which are approximately equivalent to the 1-percent seasonal exceedance temperatures. In response to RAI 2.3.1-19, the applicant has committed to updating LNP COL FSAR Section 2.3.1.2.7.3, Table 2.0-201 and Table 2.3.1-210 to include the normal ambient site characteristic temperatures that correspond with the definition of the AP1000 DCD site parameter temperatures. The revised normal ambient design site characteristic temperatures remain bounded by the AP1000 DCD site parameters, thus the staff finds the applicant's response to RAI 2.3.1-19 to be acceptable. This commitment to update the FSAR is being tracked as **Confirmatory Item 2.3.1-3**.

Resolution of Confirmatory Item 2.3.1-3

Confirmatory Item 2.3.1-3 is an applicant commitment to update Section 2.3.1 of its FSAR. The staff verified that LNP COL FSAR Section 2.3.1 was appropriately updated. As a result, Confirmatory Item 2.3.1-3 is now closed.

2.3.1.4.3 Effects of Global Climate Change on Regional Climatology

The applicant presented a discussion on the potential effects of global climate change on the regional climatology of the site. The applicant stated that even the most reliable climate models are not capable of accurately predicting design-basis extremes in weather patterns.

NUREG-0800, Section 2.3.1, states that historical data used to characterize a site should extend over a significant time interval to capture cyclical extremes. During the course of the technical review, the staff made an effort to obtain the longest period of data available to determine the adequacy of the applicant's proposed site characteristics. For example, snow load was based on a 100-year return period, ambient design temperatures were based on a minimum of 65 years of hourly data and an estimated 100-year return period value. Tornado statistics were based on a 57.25 year period and tornado wind speeds were based on a 10^{-7} per year return interval as stated in DG-1143. Extreme winds were based on a 100-year return period, including 157 years of historical hurricane data (1851-2008).

The U.S. Global Change Research Program (USGCRP) released a report to the President and Members of Congress in June 2009 entitled "Global Climate Change Impacts in the United States." This report, produced by an advisory committee chartered under the Federal Advisory Committee Act, summarizes the science of climate change and the impacts of climate change on the United States.

The USGCRP report found that the average annual temperature of the Southeast (which includes the Florida coastline where the LNP site is located) did not change significantly over the past century as a whole, but the annual average temperature has risen about 1.6 °F since 1970 with the greatest seasonal increase in temperature occurring during the winter months. Climate models predict continued warming in all seasons across the Southeast and an increase in the rate of warming through the end of the 21st century. Average temperatures in the Southeast are projected to rise by 2 – 5 °F by the end of the 2050s, depending on assumptions regarding global greenhouse gas emissions.

The USGCRP report also states that there is a 10- to 15-percent decrease in observed annual average precipitation from 1958 to 2008 in the region where the LNP site is located. Future changes in total precipitation are more difficult to project than changes in temperature. Model projections of future precipitation generally indicated that southern areas of the United States will become drier. Except for indications that the amount of rainfall from individual hurricanes will increase, climatic models provide divergent results for future precipitation for most of the Southeast.

The USGCRP reports that the power and frequency of Atlantic hurricanes has increased substantially in recent decades, but the number of North American mainland land-falling hurricanes does not appear to have increased over the past century. The USGCRP reports that likely future changes for the United States and surrounding coastal waters include more intense hurricanes with related increases in wind and rain, but not necessarily an increase in the number of these storms that make landfall.

The USGCRP further states that there is no clear trend in the frequency or strength of tornadoes since the 1950s for the United States as a whole. The applicant stated that the number of recorded tornado events has generally increased since detailed records were routinely kept beginning around 1950. However, some of this increase is attributable to a growing population, greater public awareness and interest, and technological advances in detection. The USGCRP reaches the same conclusion.

The USGCRP reports that the distribution by intensity for the strongest 10 percent of hail and wind reports is little changed, providing no evidence of an observed increase in the severity of such events. Climate models project future increases in the frequency of environmental conditions favorable to severe thunderstorms. But the inability to adequately model the small-scale conditions involved in thunderstorm development remains a limiting factor in projecting the future character of severe thunderstorms and other small-scale weather phenomena.

There is a level of uncertainty in projecting future conditions because the assumptions regarding the future level of emissions of heat trapping gases depend on projections of population, economic activity, and choice of energy technologies. If it becomes evident that long-term climatic change is influencing the most severe natural phenomena reported at the site, the COL holders have a continuing obligation to ensure that their plants stay within the licensing basis.

2.3.1.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.3.1.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to regional climatology, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

COL Information Item 2.3-1 states that a COL applicant shall address the site-specific regional climatology information. As set forth above, the applicant has presented and substantiated information to establish the regional meteorological characteristics. The staff has reviewed the information provided and for the reasons given above, concludes that the applicant has established the meteorological characteristics at the site and in the surrounding area acceptable to meet the requirements of 10 CFR 100.20(c)(2) and 10 CFR 100.21(d) with respect to determining the acceptability of the site. The staff finds that the applicant has provided a sufficient description to adequately address COL Information Item 2.3-1 (COL Action Item 2.3.1-1).

The staff finds that the applicant has considered the most severe natural phenomena historically reported for the site and surrounding area in establishing the site characteristics. Specifically, the staff has accepted the methodologies used to analyze these natural phenomena and determine the severity of the weather phenomena reflected in these site characteristics. Because the applicant has correctly implemented these methodologies, as described above, the staff has determined that the applicant has considered these historical phenomena with margin sufficient for the limited accuracy, quantity, and period of time in which the data have been accumulated in accordance with 10 CFR 52.79(a)(1)(iii).

2.3.2 Local Meteorology

2.3.2.1 *Introduction*

Section 2.3.2, "Local Meteorology," of the LNP COL FSAR addresses the local (site) meteorological parameters, the assessment of the potential influence of the proposed plant and its facilities on local meteorological conditions and the impact of these modifications on plant design and operation, and a topographical description of the site and its environs.

2.3.2.2 *Summary of Application*

Section 2.3 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.3 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.3, the applicant provided the following:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.3-2

The applicant provided additional information in LNP COL 2.3-2 to address COL Information Item 2.3-2 (COL Action Item 2.3.2-1). LNP COL 2.3-2 addresses the provision of local meteorology.

2.3.2.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for local meteorology are given in Section 2.3.2 of NUREG-0800.

The applicable regulatory requirements for identifying local meteorology are:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and time in which the historical data have been accumulated.

- 10 CFR 100.20(c)(2), and 10 CFR 100.21(d) with respect to the consideration given to the local meteorological characteristics of the site.

The related acceptance criteria from Section 2.3.2 of NUREG-0800 are as follows:

- Local summaries of meteorological data based on onsite measurements in accordance with RG 1.23, "Meteorological Monitoring Programs for Nuclear Power Plants," Revision 1, and NWS station summaries or other standard installation summaries from appropriate nearby locations (e.g., within 80-km (50-mi)) should be presented as specified in RG 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)," Section 2.3.2.1.

- A complete topographical description of the site and environs out to a distance of 80-km (50-mi) from the plant, as described in RG 1.206, Section 2.3.2.2, should be provided.

- A discussion and evaluation of the influence of the plant and its facilities on the local meteorological and air quality conditions should be provided. Applicants should also identify potential changes in the normal and extreme values resulting from plant construction and operation. The acceptability of the information is determined through comparison with standard assessments.

- The description of local site airflow should include wind roses and annual joint frequency distributions of wind speed and wind direction by atmospheric stability for all measurement levels using the criteria provided in RG 1.23, Revision 1.

2.3.2.4 *Technical Evaluation*

The NRC staff reviewed Section 2.3.2 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to local meteorology. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information contained in the LNP COL FSAR.

AP1000 COL Information Item

- LNP COL 2.3-2

The applicant provided information in LNP COL 2.3-2 to resolve COL Information Item 2.3-2, which addresses the provision of local meteorology.

The NRC staff reviewed LNP COL 2.3-2, related to the provision of local meteorology included under Section 2.3 of the LNP COL FSAR. COL Information Item 2.3-2 in Section 2.3.6.2 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will address site-specific local meteorology information.

2.3.2.4.1 Normal and Extreme Values of Meteorological Parameters

2.3.2.4.1.1 Wind Summaries

The applicant produced monthly and annual wind summaries from the onsite meteorological data from February 1, 2007 through January 31, 2009. LNP COL FSAR Tables 2.3.2-201 through 2.3.2-241 presented the average joint frequency distribution of wind speed and direction by Pasquill Stability Category (i.e., stability class) for both the lower-level (10-m) and upper-level (60-m) measurement heights. The 2-year joint frequency distribution, based on the lower-level measurement height, was used as input to the atmospheric dispersion models discussed in LNP COL FSAR Sections 2.3.4 and 2.3.5. Using the hourly meteorological data provided by the applicant, the staff independently produced the 2-year joint frequency distributions at both the lower-level and upper-level measurement heights and has confirmed the applicant's wind summaries as correct and acceptable.

Graphical illustrations of the wind summaries (i.e., wind roses) from the 1-year period February 1, 2007, through January 31, 2008, were also produced by the applicant in LNP COL FSAR Figures 2.3.2-201 through 2.3.2-213. These figures show the average monthly wind speed and direction for 16 radial compass directions over all stability classes during the 1-year period of record. Although the wind roses only include data for 1 year, the staff has confirmed that the wind speed and wind direction frequencies for the two year period from February 1, 2007, through January 31, 2009, are very similar. Using the hourly meteorological data provided by the applicant, the staff independently produced the same wind roses and has confirmed the applicant's figures as correct and acceptable.

The applicant compared the onsite wind summaries against wind speed and direction from the Tallahassee, Gainesville, and Tampa, Florida stations. The 1-year onsite wind rose provided in LNP COL FSAR Figure 2.3.2-201 shows a higher frequency of east-west winds. This pattern is also depicted in the wind roses from Gainesville and Tampa, Florida. The applicant suggests that this is most likely due to the diurnal influence of the sea breeze effects. The Tallahassee, Florida wind roses show that north-south wind patterns are most frequent. This is also most likely due to the diurnal sea breeze effects generated from the east-west directed coastline along the panhandle of Florida.

LNP COL FSAR Table 2.3.2-208 shows that the total number hours identified as calm winds at the 10-meter wind level is 3223, which is 18.8 percent of the total observations reported during the period of February 1, 2007 through January 31, 2009. In RAI 2.3.2-1, the staff asked the applicant to explain this high frequency of calm winds. In response to RAI 2.3.2-1, the applicant explained that the conditions reported as calm were for wind speeds less than the manufacturer's stated sensitivity threshold for the instrument (0.4 meters per second (m/s)). The number of calm winds at the 60-meter level drops to 174 hours, or 1.04 percent, which is considerably less than at the 10-meter level. The applicant states that the calm wind speeds at the lower level can be attributed to the height of the surrounding forest canopy, and its corresponding influence on wind speeds at the 10-meter level. The staff accepts this explanation as reasonable. Therefore, RAI 2.3.2-1 is resolved. Through the use of the 2009 ASHRAE database, the staff determined that the percentage of 10-meter level calm winds at the LNP site was comparable to the percentage of calms recorded at the five surrounding NWS recording stations.

2.3.2.4.1.2 Ambient Temperature

The applicant provided, in LNP COL FSAR Table 2.3.2-241, an ambient temperature summary based on the onsite meteorological data collected from February 1, 2007, through January 31, 2008, and five surrounding weather reporting stations (Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville, Florida). Although LNP COL FSAR Table 2.3.2-241 only includes data for 1 year, the staff has confirmed that the temperature data for the two year period from February 1, 2007, through January 31, 2009, are consistent.

Using the applicant's onsite meteorological monitoring program data from the 2-year period from February 1, 2007 through January 31 2009, and independently obtained hourly data from the five surrounding NWS observation stations, the staff has confirmed the values presented in LNP COL FSAR Table 2.3.2-241 as correct and acceptable.

2.3.2.4.1.3 Dew-Point Temperature

The applicant provided, in LNP COL FSAR Table 2.3.2-242, a dew-point temperature summary based on the onsite meteorological data collected from February 1, 2007, through January 31, 2008, and five surrounding weather reporting stations (Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville, Florida). Although LNP COL FSAR Table 2.3.2-242 only includes data for 1 year, the staff has confirmed that the dew-point temperature data for the two year period from February 1, 2007, through January 31, 2009, are consistent.

Using the applicant's onsite meteorological monitoring program data from the 2-year period from February 1, 2007 through January 31 2009, and independently obtained hourly data from the five surrounding NWS observation stations, the staff has confirmed the values presented in LNP COL FSAR Table 2.3.2-242 as correct and acceptable.

2.3.2.4.1.4 Atmospheric Moisture

The applicant presented relative humidity, precipitation, and fog data summaries from the five NWS observation stations surrounding the LNP site as well as the 1-year period of record from February 1, 2007, through January 31, 2008, from the LNP onsite meteorological data.

2.3.2.4.1.4.1 Relative Humidity

Maximum relative humidity values usually occur during the early morning hours and minimum relative humidity values typically are observed in the mid-afternoon. The applicant summarized the monthly diurnal relative humidity based on data from five surrounding reporting stations (Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville) in LNP COL FSAR Table 2.3.2-243.

The staff reviewed the data listed in the NCDC LCDs for each of the five surrounding NWS observations stations to verify the relative humidity statistics presented by the applicant and discussed in the LNP COL FSAR. The staff concludes that the values presented by the applicant are correct.

2.3.2.4.1.4.2 Precipitation

LNP COL FSAR Table 2.3.2-244 compared long-term monthly and annual precipitation measurements from the five reporting stations (Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville, Florida) to the 1-year period of record of February 1, 2007, through January 31, 2008, from the LNP onsite meteorological data. The staff has independently verified that the average monthly precipitation data recorded onsite are generally consistent with the historical data recorded at the reporting stations near the LNP site. Table 2.3.2-244 shows that Tallahassee, Florida has a higher annual average amount of precipitation than the other reporting stations. However, the total annual precipitation for 2007 was similar for all five of the stations (between 38.49 and 46.09 inches). Although LNP COL FSAR Table 2.3.2-244 only contains onsite data for 1 year, the staff has confirmed that the onsite precipitation data for the two year period from February 1, 2007, through January 31 2009, are consistent.

The staff reviewed the data listed in the NCDC LCDs for each of the five surrounding NWS observations stations to verify the precipitation statistics presented by the applicant and discussed in the LNP COL FSAR. The staff concludes that the values presented by the applicant are correct.

2.3.2.4.1.5 Fog

The applicant summarized the occurrence of fog based on data from five surrounding weather reporting stations (Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville, Florida). On average, there are 43, 23, 39, 43, and 43 days per year that fog is recorded at Tampa, Gainesville, Orlando, Tallahassee, and Jacksonville, respectively. LNP COL FSAR Table 2.3.2-245 presented the average number of days of fog per month. The staff has independently confirmed the values presented in this table as correct and adequate.

The staff reviewed the data listed in the NCDC LCDs for each of the five surrounding NWS observations stations to verify the fog statistics presented by the applicant and discussed in the LNP COL FSAR. The staff concludes that the values presented by the applicant are correct.

2.3.2.4.1.6 Atmospheric Stability

The applicant classified atmospheric stability in accordance with the guidance provided in RG 1.23, Revision 1. Atmospheric stability is a critical parameter for estimating atmospheric dispersion characteristics in LNP COL FSAR Sections 2.3.4 and 2.3.5. Dispersion of effluents is greatest for extremely unstable atmospheric conditions (i.e., Pasquill stability Class A) and decreases progressively through extremely stable conditions (i.e., Pasquill stability Class G). The applicant based its stability classification on temperature change with height (i.e., delta-temperature or $\Delta T/\Delta Z$) between the 60-m and 10-m height, as measured by the LNP onsite meteorological measurements program from February 1, 2007, through January 31, 2009.

Frequency of occurrence for each stability class is one of the inputs to the dispersion models used in LNP COL FSAR Sections 2.3.4 and 2.3.5. The applicant included these data in the form of a joint frequency distribution (JFD) of wind speed and direction data as a function of stability class. A comparison of a JFD developed by the staff from the hourly data submitted by the applicant with the JFD developed by the applicant showed reasonable agreement.

The applicant used the 2-year period of record of onsite meteorological data to produce statistics on the temporal variations of atmospheric stability as shown in LNP COL FSAR Tables 2.3.2-201 through 2.3.2-240. The staff confirmed the pattern of stability classes presented by the applicant in LNP COL FSAR Section 2.3.2.1.7. This pattern showed that the most frequent stability classes were either neutral (D) or slightly stable (E). By creating a staff derived JFD of wind speed, wind direction, and atmospheric stability and comparing it against the applicant's JFD, the staff was able to independently confirm the values presented by the applicant as correct and adequate.

2.3.2.4.2 Potential Influence of the Plant and its Facilities on Local Meteorology

2.3.2.4.2.1 Topographical Description

The applicant stated that the LNP site and surrounding area is relatively flat, with no significant terrain features that will otherwise be expected to adversely or unusually impact natural dispersion downwind of the plant. In RAI 2.3.5-3, the staff asked the applicant to discuss the influence of the Gulf of Mexico and the resulting land and sea breezes on the atmospheric dispersion estimates around the plant. In its response to RAI 2.3.5-3, the applicant discussed the strong east-west wind direction that has been shown to occur in the site area. The applicant also discussed the lower wind speeds that have been documented at the 10-meter level of the meteorological tower. Due to the lower wind speeds and the strong east-west wind direction pattern, higher predictions of relative concentration (χ/Q) and relative deposition (D/Q) can be expected in the sectors with the highest wind direction frequency. The staff agrees with this assessment of influence from the Gulf of Mexico and considers RAI 2.3.5-3 to be closed. The results of the short and long term atmospheric dispersion analysis are discussed in LNP COL FSAR Sections 2.3.4 and 2.3.5. LNP COL FSAR Figure 2.3.2-222 shows topographic features within an 80-km (50-mi) radius of the LNP site. Through an NRC staff site visit (ML100780287) and United States Geological Survey (USGS) maps, the staff has independently verified the topographical assessment provided by the applicant and accepts the description as correct and adequate.

2.3.2.4.2.2 Fogging and Icing Effects Attributable to Cooling Tower Operation

Ground fogging could occur if ground elevations in the plant vicinity were comparable to expected heights of the cooling tower plumes. The applicant stated that the expected cooling towers for Units 1 and 2 are mechanical draft towers. The applicant states that ground level fogging could occur in the immediate vicinity of the mechanical draft cooling towers. However, those events would only be expected at onsite locations and under relatively cold and moist atmospheric conditions and when building wake and downwash effects have an adverse influence on the dispersion of the cooling tower plumes. Based on previous observations and cooling tower plume modeling results (details in following section of this SER), the staff agrees and accepts the applicant's discussion.

The applicant stated that there are no large safety-related plant structures or other nearby structures that are expected to be affected by icing from cooling tower plumes due to the meteorological conditions that could reasonably be expected to occur. Because of the few days (approximately 3 days per year) with ambient temperatures below freezing at the Orlando and Tampa reporting stations, the staff agrees that the threat of ice formation is sufficiently low. The staff agrees and accepts the applicant's discussion of icing effects.

2.3.2.4.2.3 Assessment of Heat Dissipation Effects on the Atmosphere

LNP Units 1 and 2 will use two mechanical draft cooling towers to dissipate heat to the atmosphere. Potential meteorological effects due to operation of the cooling towers may include enhanced ground-level fogging and icing, cloud shadowing and precipitation enhancement, and increased ground-level humidity.

The applicant used the EPA's CALPUFF computer model to evaluate cooling tower plume behavior and to estimate the frequency of occurrence and length of visible cooling tower plumes.

The staff used the Seasonal/Annual Cooling Tower Impact (SACTI) computer code for estimating the impacts from fogging, icing, and drift deposition from the operation of the mechanical draft cooling towers. The staff found that there is a minimal threat of fogging and icing in the vicinity immediately surrounding the cooling towers. The staff agrees with the applicant's statement that because the closest public road (US Highway 19) is located 1.4 km (0.9 mi) from the nearest cooling tower, additional fogging and icing is not predicted or expected to occur in the vicinity of any public roadway.

The applicant also stated that a small amount of dissolved and suspended solids may result in solid particle deposition on the surface, primarily in close proximity to the plant. The staff has determined that nearly two months of salt accumulation would result in 0.07 milligrams per cubic centimeter (mg/cm^2), which is near the upper end of the "Light Contamination Level" range defined by the Institute of Electrical and Electronic Engineers (IEEE) standard[6]. The staff believes that total accumulation reaching amounts that require mitigation is highly unlikely due to local precipitation removing any salt deposits before it reaches a level of concern.

The staff independently confirmed the information presented in this FSAR section. The staff agrees and accepts the applicant's conclusion.

2.3.2.4.3 Local Meteorological Conditions for Design and Operating Basis

Meteorological conditions for design and operating basis are discussed in LNP COL FSAR Section 2.3.1.2 and reviewed by the staff in Section 2.3.1.4.2 of this SER.

2.3.2.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.3.2.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to regional climatology and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

COL Information Item 2.3-2 states that a COL applicant shall address the site-specific local meteorological information. As set forth above, the applicant has presented and substantiated information describing the local meteorological conditions and topographic characteristics important to evaluating the adequacy of the design and siting of this plant. The staff has reviewed the information provided and for the reasons given above, concludes that the identification and consideration of the meteorological and topographical characteristics of the

[6] IEEE Guide for Application of Power Apparatus Bushings, IEEE Standard C.57.19.100-1995, Aug 1995.

site and the surrounding area are acceptable and meet the requirements of 10 CFR 100.20(c) and 10 CFR 100.21(d). The staff finds that the applicant has provided a sufficient description to adequately address COL Information Item 2.3-2 (COL Action Item 2.3.2-1).

The staff also finds that the applicant has considered the appropriate site phenomena in establishing the site characteristics. Specifically, the staff has accepted the methodologies used to determine the meteorological and topographic characteristics. Because the applicant has correctly implemented these methodologies, as described above, the staff has determined that the site characteristics, including margins, are sufficient for the limited accuracy, quantity, and period of time in which the data have been accumulated in accordance with 10 CFR 52.79(a)(1)(iii).

2.3.3 Onsite Meteorological Measurement Programs

2.3.3.1 *Introduction*

The LNP onsite meteorological measurement program addresses the need for onsite meteorological monitoring and the resulting data. The NRC staff review covers the following specific areas: (1) meteorological instrumentation, including siting of sensors, sensor type and performance specifications, methods and equipment for recording sensor output, the quality assurance program for sensors and recorders, data acquisition and reduction procedures, and special considerations for complex terrain sites; and (2) the resulting onsite meteorological database, including consideration of the period of record and amenability of the data for use in characterizing atmospheric dispersion conditions.

This section verifies that the applicant successfully implemented an appropriate onsite meteorological measurements program and that data from this program provides an acceptable basis for estimating atmospheric dispersion for design-basis accidents (DBAs) and routine releases from an AP1000 design.

2.3.3.2 *Summary of Application*

Section 2.3 of the LNP COL FSAR, Revision 9 incorporates by reference Section 2.3 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.3, the applicant provided the following:

AP1000 COL Information Item

- LNP COL 2.3-3

The applicant provided additional information in LNP COL 2.3-3 to address COL Information Item 2.3-3 (COL Action Item 2.3.3-1). LNP COL 2.3-3 addresses the onsite meteorological measurements program.

In addition, this LNP COL FSAR section addresses Interface Item 2.9 related to the onsite meteorological measurement program.

2.3.3.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the onsite meteorological measurements program are given in Section 2.3.3 of NUREG-0800.

The applicable regulatory requirements for identifying onsite meteorological measurements program are as follows:

- 10 CFR 100.20(c)(2), with respect to the meteorological characteristics of the site that are necessary for safety analysis or that may have an impact upon plant design in determining the acceptability of a site for a nuclear power plant.

- 10 CFR 100.21(c), with respect to the meteorological data used to evaluate site atmospheric dispersion characteristics and establish dispersion parameters such that: (1) radiological effluent release limits associated with normal operation can be met for any individual located offsite; and (2) radiological dose consequences of postulated accidents meet prescribed dose limits at the exclusion area boundary (EAB) and the outer boundary of the low population zone (LPZ).

- 10 CFR Part 50, “Domestic licensing of production and utilization facilities,” Appendix A, “General Design Criteria for Nuclear Power Plants,” General Design Criterion (GDC) 19, “Control Room,” with respect to the meteorological considerations used to evaluate the personnel exposures inside the control room during radiological and airborne hazardous material accident conditions.

- 10 CFR 50.47(b)(4), 10 CFR 50.47(b)(8), and 10 CFR 50.47(b)(9), as well as Section IV.E.2 of 10 CFR Part 50, Appendix E, “Emergency Planning and Preparedness for Production and Utilization Facilities,” with respect to the onsite meteorological information available for determining the magnitude and continuously assessing the impact of the releases of radiological materials to the environment during a radiological emergency.

- 10 CFR Part 50, Appendix I, “Numerical Guides for Design Objectives and Limiting Conditions for Operation to Meet the Criteria,” with respect to meteorological data used in determining the compliance with numerical guides for design objectives and limiting conditions for operation to meet the requirement that radioactive material in effluents released to unrestricted areas be kept as low as is reasonably achievable (ALARA).

- 10 CFR Part 20, "Standards for Protection Against Radiation," Subpart D, "Radiation Dose Limits for Individual Members of the Public," with respect to the meteorological data used to demonstrate compliance with dose limits for individual members of the public.

The following RG is applicable to this section:

- RG 1.23, "Meteorological Monitoring Programs for Nuclear Power Plants," Revision 1

The related acceptance criteria from Section 2.3.3 of NUREG-0800 are as follows:

- The preoperational and operational monitoring programs should be described, including: (1) a site map (drawn to scale) that shows tower location and true north with respect to man-made structures, topographic features, and other features that may influence site meteorological measurements; (2) distances to nearby obstructions of flow in each downwind sector; (3) measurements made; (4) elevations of measurements; (5) exposure of instruments; (6) instrument descriptions; (7) instrument performance specifications; (8) calibration and maintenance procedures and frequencies; (9) data output and recording systems; and (10) data processing, archiving, and analysis procedures.

- Meteorological data should be presented in the form of JFDs of wind speed and wind direction by atmospheric stability class in the format described in RG 1.23, Revision 1. An hour-by-hour listing of the hourly-averaged parameters should be provided in the format described in RG 1.23, Revision 1. If possible, evidence of how well these data represent long-term conditions at the site should also be presented, possibly through comparison with offsite data.

- At least two consecutive annual cycles (and preferably 3 or more whole years), including the most recent 1-year period, should be provided with the application. These data should be used by the applicant to calculate: (1) the short-term atmospheric dispersion estimates for accident releases discussed in SER Section 2.3.4; and (2) the long-term atmospheric dispersion estimates for routine releases discussed in SER Section 2.3.5.

The applicant should identify and justify any deviations from the guidance provided in RG 1.23, Revision 1. Deviations from guidance are discussed in further detail in Chapter 1 of this SER.

2.3.3.4 *Technical Evaluation*

The NRC staff reviewed Section 2.3.3 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL applications represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the onsite meteorological measurements program. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

AP1000 COL Information Item

- LNP COL 2.3-3

The NRC staff reviewed LNP COL 2.3-3 related to the onsite meteorological measurements program included under Section 2.3 of the LNP COL FSAR. The specific text of this COL information item in Section 2.3.6.3 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will address the site-specific onsite meteorological measurements program.

The staff's evaluation is based on the descriptions provided by the applicant in LNP COL FSAR Section 2.3.3 and a pre-application readiness assessment held November 3-7, 2008. The purpose of the readiness assessment was to: (1) become familiar with the prospective applicant's site and site selection process, plans, schedules, and initiatives; (2) observe and review the preoperational onsite meteorological monitoring program; and (3) review the prospective applicant's plans for its operational onsite meteorological monitoring program.

The NRC staff relied upon the review procedures presented in NUREG-0800, Section 2.3.3, to independently assess the technical sufficiency of the information presented by the applicant.

2.3.3.4.1 Preoperational Meteorological Measurement Program

The onsite meteorological measurements program at the LNP site began in February 2007. LNP COL FSAR Figure 2.3.3-201 shows the location of the meteorological tower with respect to LNP Units 1 and 2 along with the topography of the site. RG 1.23, Revision 1, Section 3 describes an acceptable method for siting of the onsite meteorological observation tower. The meteorological tower is a 60.4-m guyed, open-latticed meteorological tower located at an elevation of approximately 13.7 m. The largest structures in the vicinity that have the potential to influence the meteorological measurements are the mechanical draft cooling towers. RG 1.23, Revision 1, indicates that obstructions to flow (such as buildings) should be located at least 10 obstruction heights from the meteorological tower to prevent adverse building wake effects. The height of the closest mechanical draft cooling tower is 17.1 m. The LNP meteorological tower, located approximately 600 m from the nearest cooling tower, is of a sufficient distance. The tower design is consistent with the guidance provided in RG 1.23, Revision 1; therefore, it is acceptable to the staff.

Due to the relatively short distance between the mechanical draft cooling towers and the onsite meteorological tower, the staff was concerned that moist plumes from the cooling towers could potentially affect the measurements taken at the meteorology tower. In RAI 2.3.3-7, the staff requested that the applicant provide a discussion in the LNP COL FSAR on the potential effects of the cooling tower plumes on the onsite meteorological system measurements during plant operation. In response to RAI 2.3.3-7, the applicant provided a discussion on the frequency in which the meteorological measurements system may be impacted by plumes from the mechanical draft cooling towers. The applicant stated that visible plumes greater than 500 m in length are expected to occur less than 3 percent of the time. Winds from the northeast (the direction from cooling towers to the meteorological tower) have been observed to occur 9 percent of the time during the 2-year period from February 1, 2007, through January 31, 2009. Therefore, a visible plume from the cooling towers extending to the meteorological monitoring tower would occur less than 0.3 percent of the time. Through cooling tower plume modeling and analysis of the hourly onsite meteorological dataset, the staff has confirmed the information provided in the RAI response and the discussion and accepts it as correct and adequate. Therefore, the staff considers the clarifications requested in RAI 2.3.3-7 to be resolved. The applicant has agreed to update the FSAR to include this information. Therefore, LNP COL FSAR Section 2.3.3.1 is being tracked as **Confirmatory Item 2.3.3-1**.

Resolution of Confirmatory Item 2.3.3-1

Confirmatory Item 2.3.3-1 is an applicant commitment to update section 2.3.3.1 of its FSAR. The staff verified that LNP COL FSAR Section 2.3.3.1 was appropriately updated. As a result, Confirmatory Item 2.3.3-1 is now closed.

Also, the applicant describes the area surrounding the tower as relatively flat. Moreover, the area is flat within several miles of the site with no appreciable or noticeable variation in the terrain that would have a significant influence on the observed meteorological parameters. Through the use of USGS maps, the staff as confirmed the information presented in the LNP COL FSAR in regards to the topography surrounding the meteorological tower. In the immediate vicinity of the tower base and within the security fence, gravel has been used as a means of controlling weeds. The presence of this gravel is not extensive and is not expected to have an influence on the measured parameters. The staff confirmed the description of the area immediately surrounding the meteorological tower during an NRC staff site visit (ML100780287).

LNP COL FSAR Table 2.3.3-201 provides the heights at which the measurements are made in the onsite meteorological measurements program. Wind speed and direction, ambient temperature, and delta-temperature are recorded at both the 60 m and 10 m levels. Dew point temperature is recorded at the 10 m level. Solar radiation, precipitation and barometric pressure are all recorded at the base of the tower at the 2.0 m level. The height of the measurements complies with the recommended instrument heights described in Section 2 of RG 1.23, Revision 1; therefore, they are acceptable to the staff.

2.3.3.4.1.1 Wind Systems

Wind direction, wind speed, and wind direction variance (i.e., sigma theta) are monitored at both the lower- (10-m) and upper-level (60-m) of the tower. The wind sensors are mounted on a 3.7-m retractable boom oriented perpendicular to the prevailing wind flow. Section 3 of RG 1.23, Revision 1, states that wind sensors should be mounted at a distance equal to at least twice the horizontal dimension of the tower. This is to reduce the possible interference of the tower structure on the wind instruments. As shown in LNP COL FSAR Table 2.3.3-202, these measurements are based on the guidance provided in Sections 2 and 3 of RG 1.23, Revision 1; therefore, the wind systems are acceptable to the staff.

2.3.3.4.1.2 Temperature Systems

Ambient temperature and delta-temperature are monitored at both the lower- and upper-level of the tower. Two channels of differential temperature are monitored simultaneously between the lower- and upper-levels. The temperature probes are mounted in aspirated shields attached to a 2.5-m retractable boom. Dew point temperature is measured at the 10-m level of the tower. Section 3 of RG 1.23, Revision 1, states that ambient temperature and moisture measurements should be made to avoid air modification by heat and moisture sources. As shown in LNP COL FSAR Table 2.3.3-202, the temperature systems are based the guidance provided in Sections 2 and 3 of RG 1.23, Revision 1; therefore the temperature systems are acceptable to the staff.

2.3.3.4.1.3 Precipitation and Solar Radiation Systems

Precipitation and solar radiation are measured at 2.0 m above ground-level by sensors located near the base of the tower. As shown in LNP COL FSAR Table 2.3.3-202, the precipitation sensor is based on RG 1.23, Revision 1, and the solar radiation sensor is based on American National Standards Institute/American Nuclear Society (ANSI/ANS) 2.5-1984[7]. Since no accuracies are specified in RG 1.23, Revision 1 for solar radiation instrumentation, the applicant has chosen to follow the recommendations in ANSI/ANS 2.5-1984, a document endorsed by the NRC. Therefore, the precipitation and solar radiation systems are acceptable to the staff.

2.3.3.4.1.4 Maintenance and Calibration

The applicant stated that the meteorological equipment is checked and calibrated on a routine basis in accordance with RG 1.23, Revision 1. In order to achieve the required level of system reliability, as specified in RG 1.23, Revision 1, the applicant employs the following maintenance techniques: (1) calibrating the datalogger input channels semiannually; (2) calibrating or replacing the wind sensors with National Institute for Standards and Technology (NIST)-traceable calibrated sensors semiannually; (3) calibrating or replacing barometric pressure, dew-point temperature, and solar radiation channel sensors with NIST-traceable calibrated sensors; (4) calibrating and checking the consistency between the two ambient/differential temperature channels; (5) checking the guyed wires and the tower anchors annually.

In RAI 2.3.3-8, the staff requested that the applicant update the LNP COL FSAR to clarify how often the onsite meteorological temperature sensors (thermistors) are calibrated and replaced. In response to RAI 2.3.3-8, the applicant stated that LNP COL FSAR Section 2.3.3.1.4 will be updated to clarify that the thermistors are calibrated every 6 months to ensure proper sensor

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[7] ANS, 1984. Standard for Determining Meteorological Information at Nuclear Power Sites. ANSI/ANS-2.5-1984. American Nuclear Society, La Grange Park, IL.

operation. This commitment to update the FSAR is being tracked as **Confirmatory Item 2.3.3-2**.

Resolution of Confirmatory Item 2.3.3-2

Confirmatory Item 2.3.3-2 is an applicant commitment to update Section 2.3.3 of its FSAR. The staff verified that LNP COL FSAR Section 2.3.3 was appropriately updated. As a result, Confirmatory Item 2.3.3-2 is now closed.

The staff finds that the instrument maintenance and calibration techniques are consistent with the guidance provided in RG 1.23, Revision 1; therefore, they are acceptable to the staff.

2.3.3.4.1.5 Data Reduction

The applicant described in Section 2.3.3.1.5 of the LNP COL FSAR that data from the datalogger, located near the meteorological tower, are retrieved via remote connection through a cellular telephone link. Using a host computer, an offsite meteorological consultant retrieves the meteorological data from the datalogger on a daily basis (except weekends and holidays). The data is compared against data from an Automatic Weather Observing System operated by the municipality of Ocala, as well as data from the nearby Crystal River Energy Complex. Erroneous data are discarded prior to being saved in the historical database. The edited and reviewed 15-minute averaged data are then stored on electronic media.

RG 1.23, Revision 1, states that: (1) the digital sampling rate for data should be at least once every 5 seconds; (2) digital data should be compiled as 15-minute average values for real-time display in the appropriate emergency response facilities; and (3) digital data should be compiled and archived as hourly values for use in historical climatic and dispersion analyses. Data should also be compiled into annual JFDs of wind speed and wind direction by atmospheric stability class.

The following information, summarized from Section 2.3.3.1.5 of the LNP COL FSAR, is the routine output as part of the onsite measurements program:

- Temperature, pressure, precipitation, solar radiation, and dew-point temperature summaries as daily and monthly averages

- Hourly precipitation totals

- Hourly averages of pressure, temperature, delta-temperature, dew-point temperature, upper- and lower-level wind direction and wind speed, upper- and lower-level wind direction variance, Pasquill stability classes, and accumulated solar radiation

- 15-minute averages of all parameters, except precipitation, which is a 15-minute total value

- Joint frequency distributions of upper- and lower-level wind speed, wind direction, and Pasquill stability class

The data reduction and compilation techniques listed above comply with the recommendations provided in RG 1.23, Revision 1 and are therefore acceptable to the staff.

2.3.3.4.1.6 Accuracy of Measurements

LNP COL FSAR Table 2.3.3-202 summarizes the accuracy of the measurements taken as part of the LNP onsite meteorological measurements program. The accuracy of the 2-year period of record for the data provided was consistent with the requirements of RG 1.23, Revision 1, with the exception of the dew-point temperature measurements, which met the requirements of NRC endorsed ANSI/ANS 2.5-1984. Therefore, the accuracy of the measurements is acceptable to the staff.

2.3.3.4.2 Operational Meteorological Measurement Program

The applicant stated that the operational meteorological monitoring program will be a continuation of the pre-operational program. The pre-operational and operational monitoring programs are described jointly in the LNP COL FSAR. Since the pre-operational monitoring program meets the guidance provided in RG 1.23, Revision 1, the staff finds the continuation of this program to be acceptable.

2.3.3.4.3 Meteorological Data

As discussed in SER Section 2.3.2.4.1.1, the applicant provided JFDs of wind speed, wind direction, and atmospheric stability for both the 10-meter and 60-meter levels based on hourly measurements taken from February 1, 2007, through January 31, 2009.

The staff performed a quality review of the 2007-2009 hourly meteorological database using the methodology described in NUREG-0917, “Nuclear Regulatory Commission Staff Computer Programs for Use with Meteorological Data,” issued July 1982. The staff used computer spreadsheets to perform further review. As expected, the staff’s examination of the data revealed generally stable and neutral atmospheric conditions at night and unstable conditions during the day. Wind speed, wind direction, and stability class frequency distributions for each measurement channel were reasonable. As discussed in SER Section 2.3.2.4.1.1, the staff verified and accepts the lower- and upper-level JFDs and wind roses provided by the applicant.

In order to evaluate the accuracy of the 2007-2009 dataset, the staff compared the hourly temperature measurements to the observation sites at Jacksonville, Tallahassee, and Tampa. These comparisons showed agreeing patterns with the surrounding weather reporting sites. Based on these results, the staff believes that the data collected is acceptable and representative of site conditions.

Based on an independent quality review of the onsite meteorological data and a comparison with off-site weather reporting station data, the staff accepts the 2-years of onsite data provided by the applicant. The staff has determined that the data is representative of the site and is an acceptable basis for estimating atmospheric dispersion for accidental and routine releases in LNP COL FSAR Sections 2.3.4 and 2.3.5.

2.3.3.5 *Post Combined License Activities*

Part 10 of the LNP COL application describes proposed COL conditions, including inspection, test, analysis, and acceptance criteria (ITAAC). Table 3.8-1 in Part 10 of the COL application includes the emergency planning (EP) ITAAC. The following EP ITAAC involve demonstrating that the operational onsite meteorological monitoring program appropriately supports the LNP emergency plan:

- EP Program Element 7.1: The licensee has established a technical support center (TSC), which receives, stores, processes, and displays plant and environmental information, and enables the initiation of emergency measures and the conduct of emergency assessment (Acceptance Criteria 7.1.5).

- EP Program Element 7.2: The licensee has established an emergency operating facility in which meteorological data is acquired, displayed, and evaluated pertinent to offsite protective measures (Acceptance Criteria 7.2.2).

- EP Program Element 7.6: The means exists to provide meteorological information, consistent with Appendix 2 of NUREG-0654/FEMA-REP-1, "Criteria for Preparation and Evaluation of Radiological Emergency Response Plans and Preparedness in Support of Nuclear Power Plants," Revision 1. LNP meteorological equipment will be able to assess and monitor actual or potential offsite consequences of a radiological condition related to atmospheric measurements (Acceptance Criteria 7.6).

- EP Program Element 8.3: The means exists to continuously assess the impact of the release of radioactive materials to the environment, accounting for the relationship between effluent monitor readings, and onsite and offsite exposures and contamination for various meteorological conditions.

- EP Program Element 8.4: The means exists to acquire and evaluate meteorological information.

EP, including EP ITAAC are addressed in SER Section 13.3, "Emergency Planning."

2.3.3.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to the onsite meteorological measurements program, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

COL Information Item 2.3-3 states that a COL applicant shall address the site-specific onsite meteorological measurements program. As set forth above, the applicant has presented and substantiated information pertaining to the onsite meteorological measurements program and the resulting database. The staff has reviewed the information provided in LNP COL 2.3-3. The staff concludes that the applicant has established consideration of the onsite meteorological measurements program and the resulting database are acceptable, and meet the requirements of 10 CFR 100.20 with respect to determining the acceptability of the site. The staff also finds that the onsite data also provide an acceptable basis for making estimates of atmospheric dispersion for DBA and routine releases from the plant to meet the requirements of 10 CFR 100.21, GDC 19, 10 CFR Part 20, and Appendix I to 10 CFR Part 50. Finally, the equipment provided for measurement of meteorological parameters during the course of accidents is sufficient to provide reasonable prediction of atmospheric dispersion of airborne radioactive materials in accordance with Appendix E to 10 CFR Part 50. Part 5, "Emergency Plan" of the LNP COL application identifies alternative offsite sources of meteorological data during an emergency. The staff finds that the applicant has provided a sufficient description to adequately address COL Information Item 2.3-3 (COL Action Item 2.3.3-1).

2.3.4 Short-Term Diffusion Estimates (Related to RG 1.206, Section C.III.1, Chapter 2, C.I.2.3.4, "Short-Term Atmospheric Dispersion Estimates for Accident Releases"

2.3.4.1 *Introduction*

The short-term diffusion estimates are used to determine the amount of airborne radioactive materials expected to reach a specific location during an accident situation. The diffusion estimates address the requirement for conservative atmospheric dispersion (relative concentration) factor (χ/Q value) estimates at the EAB, the outer boundary of the LPZ, and at the control room for postulated design-basis accidental radioactive airborne releases. The review covers the following specific areas: (1) atmospheric dispersion models to calculate atmospheric dispersion factors for postulated accidental radioactive releases; (2) meteorological data and other assumptions used as input to atmospheric dispersion models; (3) derivation of diffusion parameters (e.g., σ_y and σ_z); (4) cumulative frequency distributions of χ/Q values; (5) determination of conservative χ/Q values used to assess the consequences of postulated design-basis atmospheric radioactive releases to the EAB, LPZ, and control room; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52, "Licenses, certifications, and approvals for nuclear power plants."

2.3.4.2 *Summary of Application*

Section 2.3.4 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.3.4 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.3, the applicant provided the following:

AP1000 COL Information Item

- LNP COL 2.3-4

The applicant provided additional information in LNP COL 2.3-4 to address COL Information Item 2.3-4 (COL Action Item 2.3.4-1). LNP COL 2.3-4 addresses the provision of site-specific short-term diffusion estimates for NRC review to ensure that the bounding values (Table 2-1 and Appendix 15A from the AP1000 DCD) of relative concentrations are not exceeded.

In addition, this LNP COL FSAR section addresses Interface Item 2.4 related to the limiting meteorological parameters (χ/Q) for DBAs.

2.3.4.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the short-term diffusion estimates are given in Section 2.3.4 of NUREG-0800.

The applicable regulatory requirements for the applicant's description of atmospheric diffusion estimates for accidental releases are as follows:

- 10 CFR Part 50, Appendix A, GDC 19, with respect to the meteorological considerations used to evaluate the personnel exposures inside the control room during radiological and airborne hazardous material accident conditions.

- 10 CFR 52.79(a)(1)(vi) with respect to a safety assessment of the site, including consideration of major SSCs of the facility and site meteorology, to evaluate the offsite radiological consequences at the EAB and LPZ.

- 10 CFR 100.21(c)(2), with respect to the atmospheric dispersion characteristics used in the evaluation of the EAB and LPZ radiological dose consequences for postulated accidents.

The following RGs are applicable to this section:

- RG 1.78, “Evaluating the Habitability of a Nuclear Power Plant Control Room During a Postulated Hazardous Chemical Release,” Revision 1

- RG 1.145, “Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants,” Revision 1

- RG 1.194, “Atmospheric Relative Concentrations for Control Room Radiological Habitability Assessments at Nuclear Power Plants”

The related acceptance criteria from Section 2.3.4 of NUREG-0800 are as follows:

- A description of the atmospheric dispersion models used to calculate χ/Q values for accidental releases of radioactive and hazardous materials to the atmosphere.

- Meteorological data used for the evaluation (as input to the dispersion models), which represent annual cycles of hourly values of wind direction, wind speed, and atmospheric stability for each mode of accidental release

- A discussion of atmospheric diffusion parameters, such as lateral and vertical plume spread (σ_y and σ_z) as a function of distance, topography, and atmospheric conditions, should be related to measured meteorological data.

- Hourly cumulative frequency distributions of χ/Q values from the effluent release point(s) to the EAB and LPZ should be constructed to describe the probabilities of these χ/Q values being exceeded.

- Atmospheric dispersion factors used for the assessment of consequences related to atmospheric radioactive releases to the control room for design basis, other accidents and for onsite and offsite releases of hazardous airborne materials should be provided.

- For control room habitability analysis, a site plan drawn to scale should be included showing true North and potential atmospheric accident release pathways, control room intake, and unfiltered inleakage pathways.

2.3.4.4 *Technical Evaluation*

The NRC staff reviewed Section 2.3.4 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the short-term diffusion estimates. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information contained in the LNP COL FSAR:

AP1000 COL Information Item

- LNP COL 2.3-4

The NRC staff reviewed LNP COL 2.3-4 related to the short-term diffusion estimates included under Section 2.3.4 of the LNP COL FSAR. The COL Information Item 2.3-4 in Section 2.3.6.4 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will address the site-specific χ/Q values specified in subsection 2.3.4. For a site selected that exceeds the bounding χ/Q values, the Combined License applicant will address how the radiological consequences associated with the controlling design basis accident continue to meet the dose reference values given in 10 CFR Part 50.34 and control room operator dose limits given in General Design Criteria 19 using site-specific χ/Q values. The Combined License applicant should consider topographical characteristics in the vicinity of the site for restrictions of horizontal and/or vertical plume spread, channeling or other changes in airflow trajectories, and other unusual conditions affecting atmospheric transport and diffusion between the source and receptors. No further action is required for sites within the bounds of the site parameters for atmospheric dispersion.

The NRC staff relied upon the review procedures presented in NUREG-0800, Section 2.3.4, to independently assess the technical sufficiency of the information presented by the applicant.

2.3.4.4.1 Atmospheric Dispersion Models

2.3.4.4.1.1 Offsite Dispersion Estimates

The applicant used the computer code PAVAN (NUREG/CR-2858, "PAVAN: An Atmospheric Dispersion Program for Evaluating Design-Basis Accidental Releases of Radioactive Materials from Nuclear Power Stations") to estimate χ/Q values at the EAB and at the outer boundary of the LPZ for potential accidental releases of radioactive material. The PAVAN model implements the methodology outlined in RG 1.145, "Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants," Revision 1.

The PAVAN code estimates χ/Q values for various time-averaged periods ranging from 2 hours to 30 days. The meteorological input to PAVAN consists of a JFD of hourly values of wind speed and wind direction by atmospheric stability class. The χ/Q values calculated through PAVAN are based on the theoretical assumption that material released to the atmosphere will be normally distributed (Gaussian) about the plume centerline. A straight-line trajectory is assumed between the point of release and all distances for which χ/Q values are calculated.

For each of the 16 downwind direction sectors (e.g., N, NNE, NE, ENE), PAVAN calculates χ/Q values for each combination of wind speed and atmospheric stability at the appropriate downwind distance (i.e., the EAB and the outer boundary of the LPZ). The χ/Q values calculated for each sector are then ordered from greatest to smallest and an associated cumulative frequency distribution is derived based on the frequency distribution of wind speed and stabilities for each sector. The smallest χ/Q value in a distribution will have a corresponding cumulative frequency equal to the wind direction frequency for that particular sector. PAVAN determines for each sector an upper envelope curve based on the derived data (plotted as χ/Q versus probability of being exceeded), such that no plotted point is above the curve. From this upper envelope, the χ/Q value, which is equaled or exceeded 0.5 percent of the total time, is obtained. The maximum 0.5 percent χ/Q value from the 16 sectors becomes the 0-2 hour "maximum sector χ/Q value."

Using the same approach, PAVAN also combines all χ/Q values independent of wind direction into a cumulative frequency distribution for the entire site. An upper envelope curve is determined, and the program selects the χ/Q value, which is equaled or exceeded 5.0 percent of the total time. This is known as the 0-2 hour "5-percent overall site χ/Q value."

The larger of the two χ/Q values, either the 0.5-percent maximum sector value or the 5-percent overall site value, is selected to represent the χ/Q value for the 0–2 hour time interval (note that this resulting χ/Q value is based on 1-hour averaged data but is conservatively assumed to apply for 2 hours). An alternative method to determine the χ/Q value for the 0-2 hour time interval is to retain the maximum possible χ/Q value based on the distance, calm wind speeds, and G-stability.

To determine χ/Q values for longer time periods (i.e., 0-8 hour, 8-24 hour, 1-4 days, and 4-30 days), PAVAN performs a logarithmic interpolation between the 0-2 hour χ/Q values and the annual average (8760-hour) χ/Q values for each of the 16 sectors and overall site. For each time period, the highest among the 16 sectors and overall site χ/Q values is identified and becomes the short-term site characteristic χ/Q value for that time period.

In RAI 2.3.4-4, the staff requested that the applicant remove LNP COL FSAR Table 2.3.4-205 and the discussion associated with the 50 percent EAB and LPZ χ/Q values because they are discussed in the Environmental Report and are not compared against any AP1000 DCD site parameter. The applicant agreed to remove the table and the discussion from the FSAR. RAI 2.3.4-4 is, therefore, resolved. This agreement to update the FSAR is being tracked as **Confirmatory Item 2.3.4-1**.

Resolution of Confirmatory Item 2.3.4-1

Confirmatory Item 2.3.4-1 is an applicant commitment to update Section 2.3.4 of its FSAR. The staff verified that LNP COL FSAR Section 2.3.4 was appropriately updated. As a result, Confirmatory Item 2.3.4-1 is now closed.

2.3.4.4.1.2 Control Room Dispersion Estimates

The applicant used the computer code ARCON96 (NUREG/CR-6331, "Atmospheric Relative Concentrations in Building Wakes") to estimate χ/Q values at the control room for potential accidental releases of radioactive material. The ARCON96 model implements the methodology outlined in RG 1.194, "Atmospheric Relative Concentrations for Control Room Radiological Habitability Assessments at Nuclear Power Plants."

The ARCON96 code estimates χ/Q values for various time-averaged periods ranging from 2 hours to 30 days. The meteorological input to ARCON96 consists of hourly values of wind speed, wind direction, and atmospheric stability class. The χ/Q values calculated through ARCON96 are based on the theoretical assumption that material released to the atmosphere will be normally distributed (Gaussian) about the plume centerline. A straight-line trajectory is assumed between the release points and receptors. The diffusion coefficients account for enhanced dispersion under low wind speed conditions and in building wakes.

The hourly meteorological data are used to calculate hourly relative concentrations. The hourly relative concentrations are then combined to estimate concentrations ranging in duration from 2 hours to 30 days. Cumulative frequency distributions, prepared from the average relative concentrations and the relative concentrations that are exceeded no more than five percent of the time for each averaging period, are determined.

2.3.4.4.2 Meteorological Data Input

2.3.4.4.2.1 Offsite Dispersion Estimates

The meteorological input to PAVAN used by the applicant consisted of a JFD of wind speed, wind direction, and atmospheric stability based on hourly onsite data from a 2-year period from February 1, 2007 through January 31, 2009. The wind data were obtained from the 10-m level of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-temperature) measurements taken between the 60-m and 10-m levels on the onsite meteorological tower.

The staff has completed a detailed review related to the acceptability and representativeness of the hourly meteorological data as discussed in SER Sections 2.3.2 and 2.3.3. Based on this review, the staff considers the onsite meteorological database suitable for input to the PAVAN model.

2.3.4.4.2.2 Control Room Dispersion Estimates

The meteorological input to ARCON96 used by the applicant consisted of wind speed, wind direction, and atmospheric stability data based on hourly onsite data from a 2-year period from February 1, 2007, through January 31, 2009. The wind data were obtained from the 10-m and 60-m levels of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-temperature) measurements taken between the 60-m and 10-m levels on the onsite meteorological tower.

The staff has completed a detailed review related to the acceptability and representativeness of the hourly meteorological data as discussed in SER Section 2.3.3. Based on this review, the staff considers the onsite meteorological database suitable for input to the ARCON96 model.

2.3.4.4.3 Diffusion Parameters

2.3.4.4.3.1 Offsite Dispersion Estimates

The applicant chose to implement the diffusion parameter assumptions outlined in RG 1.145, Revision 1, as a function of atmospheric stability for its PAVAN model runs. The staff evaluated the applicability of the PAVAN diffusion parameters and concluded that no unique topographic features (such as rough terrain, restricted flow conditions, or coastal or desert areas) preclude the use of the PAVAN model for the LNP site. In RAI 2.3.5-3, the staff asked the applicant to discuss the influence of the Gulf of Mexico and the resulting land and sea breezes on the atmospheric dispersion estimates. The applicant responded by explaining that the daily interchanges of onshore and offshore flow directions appear to be contributing to low average wind speed. In general, the location of the LNP site would be expected to result in higher predictions of χ/Q values due to the lower wind speeds or to an increase in the frequency of wind directions in specific sectors. Based on an independent analysis of the short-term dispersion estimates, the staff accepts the applicant's description.

Therefore, the staff finds that the applicant's use of diffusion parameter assumptions, as outlined in RG 1.145, Revision 1 acceptable.

2.3.4.4.3.2 Control Room Dispersion Estimates

The diffusion coefficients used in ARCON96 and incorporated by the applicant have three components. The first component is the diffusion coefficient used in other NRC models such as PAVAN. The other two components are corrections to account for enhanced dispersion under low wind speed conditions and in building wakes. These components are based on analysis of diffusion data collected in various building wake diffusion experiments under a wide range of meteorological conditions. Because the diffusion occurs at short distances within the plant's building complex, the ARCON96 diffusion parameters are not affected by nearby topographic features such as bodies of water. Therefore, the staff finds the applicant's use of the ARCON96 diffusion parameter assumptions acceptable.

2.3.4.4.4 Relative Concentration for Accident Consequences Analysis

2.3.4.4.4.1 Conservative Short-Term Atmospheric Dispersion Estimates for EAB and LPZ

The applicant modeled one ground-level release point and used the AP1000 DCD dimensions (AP1000 DCD Figure 3.8.2-1) for the minimum building cross section and containment heights for building wake effects. Including the building wake effects for a ground-level release has little influence on the predicted χ/Q values. A ground-level release assumption that assumes the appropriate building dimensions is acceptable to the staff. This is acceptable because the PAVAN model includes both plume meander and building wake effects, which are mutually exclusive. As discussed in LNP COL FSAR Section 2.1 the EAB and LPZ are defined as two overlapping circles centered on the reactor building of each unit.

While performing a confirmatory analysis of the χ/Q values for ground level releases, the staff questioned the applicant's use of a significant number of calm or light wind conditions in their PAVAN model run. In RAI 2.3.4-5, the staff requested that the applicant follow the guidance provided in Section C.1.1.1 of RG 1.145, which states that if the meteorological instrumentation conforms to RG 1.23 (i.e., if the wind sensors have a starting threshold less than 0.45 m/s) then calms should be assigned a wind speed equal to the vane or anemometer starting speed, whichever is higher. In response to this RAI, the applicant updated its methodology to include a JFD with lower wind speed classes to represent calms in accordance with meteorological instrumentation limits, as recommended in RG 1.145, Revision 1. The applicant also updated the χ/Q results to reflect the maximum possible sector-dependent 2-hour χ/Q values instead of extrapolated 0.5 percent sector-dependent χ/Q values from PAVAN. The highest 2-hour χ/Q values typically occur under stability Class G (extremely stable) conditions and at low wind speeds; the applicant generated its maximum possible sector-dependent 2-hour χ/Q values assuming G stability and a wind speed less than the wind sensor starting threshold of 0.45 m/s. The staff independently confirmed the applicant's results, and accepts the content of the RAI response as correct and adequate; therefore, RAI 2.3.4-5 is closed. The staff also finds that the LNP COL FSAR revised site characteristics in Table 2.0-201 remain bounded by the AP1000 DCD site parameters. The commitment to update the FSAR Section 2.3.4 text as well as FSAR Tables 2.3.4-201 through 2.3.4-204, and 2.0-201 to reflect the updated JFD and the maximum possible sector χ/Qs is being tracked as **Confirmatory Item 2.3.4-2**.

Resolution of Confirmatory Item 2.3.4-2

Confirmatory Item 2.3.4-2 is an applicant commitment to update Section 2.3.4 of its FSAR as well as FSAR Tables 2.3.4-201 through 2.3.4-204, and 2.0-201 to reflect the updated JFD and the maximum possible sector χ/Qs. The staff verified that LNP COL FSAR Section 2.3.4 and FSAR Tables 2.3.4-201, 2.3.4-202, 2.3.4-203, 2.3.4-204, and 2.0-201 were appropriately updated. As a result, Confirmatory Item 2.3.4-2 is now closed. LNP COL FSAR Table 2.3.4-201 compared the site-specific EAB and LPZ χ/Q values to the corresponding site parameters provided in AP1000 DCD Table 2-1. This comparison showed that the AP1000 DCD EAB and LPZ χ/Q values bounded the revised site-specific values provided by the applicant in its response to RAI 2.3.4-5.[8]

Using the information provided by the applicant, including the 10-m level joint frequency distributions of wind speed, wind direction, and atmospheric stability presented in LNP COL FSAR Tables 2.3.2-201 through 2.3.2-208, the staff confirmed the applicant's χ/Q values by running the PAVAN computer code and obtaining consistent results. The applicant's joint frequency distributions used twelve wind speed categories. These wind speed categories were based on RG 1.23, Revision 1, but included an additional category to correspond with the manufacturer's stated instrument threshold wind speed. The staff accepts the short-term χ/Q values presented by the applicant.

[8] Smaller χ/Q values are associated with greater dilution capability, resulting in lower radiological doses. When comparing a DCD site parameter χ/Q value and a site characteristic χ/Q value, the site is acceptable for the design if the site characteristic χ/Q value is smaller than the site parameter χ/Q value. Such a comparison shows that the site has better dispersion characteristics than that required by the reactor design.

2.3.4.4.4.2 Short-Term Atmospheric Dispersion Estimates for the Control Room

The applicant provided the following as the necessary input to ARCON96:

- Onsite Hourly Meteorological Data: February 1, 2007 – January 31, 2009
- AP1000 DCD Table 15A-7: Control Room Source / Receptor Data
- AP1000 DCD Figure 15A-1: Site Plan with Release and Intake Locations
- LNP COL FSAR Table 2.3.4-207: Release / Receptor Azimuthal Angles
- LNP COL FSAR Figure 2.1.1-203: Plant Layout on the LNP Site

Two receptor (i.e., air intake) points, the control room heating, ventilation, and air conditioning (HVAC) intake and control room door, were modeled for the following eight release points:

- Containment Shell
- Fuel Building Blowout Panel
- Fuel Building Rail Bay Door
- Steam Vent
- Power-Operated Relief Valve (PORV) / Safety Valves
- Condenser Air Removal Stack
- Plant Vent
- PCS Air Diffuser

LNP COL FSAR Table 2.3.4-206 compared the site-specific control room χ/Q values to the corresponding site parameters provided in the DCD. This comparison showed that the AP1000 control room χ/Q values conservatively bounded the site-specific values. This comparison is reproduced in LNP COL FSAR Table 2.0-201.

The staff confirmed the applicant's atmospheric dispersion estimates by running the ARCON96 computer model and obtaining similar results (i.e., values on average within ± 5.2 percent). Both the staff and applicant used a ground-level release assumption for each of the release/receptor combinations as well as other conservative assumptions. Based on its confirmatory analysis, the staff finds the applicant's control room χ/Q values acceptable.

2.3.4.4.5 Onsite and Offsite Hazardous Materials

A review of the identification of onsite and offsite hazardous materials that could threaten control room habitability is performed in SER Sections 2.2.1, 2.2.2, and 2.2.3. The accident scenarios, including release characteristics and atmospheric dispersion model descriptions are also found in these sections.

2.3.4.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.3.4.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to short-term diffusion estimates, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

COL Information Item 2.3-4 states that a COL applicant shall address the site-specific χ/Q values as specified in AP1000 DCD Section 2.3.4. The staff concludes that the applicant's atmospheric dispersion estimates are acceptable and meet the relevant requirements of 10 CFR 100.21(c)(2). This conclusion is based on the conservative assessments of post-accident atmospheric dispersion conditions that have been made by the applicant and the staff from the applicant's meteorological data and appropriate diffusion models.

These atmospheric dispersion estimates are appropriate for the assessment of consequences from radioactive releases for DBAs in accordance with 10 CFR 52.79(a)(1)(vi), 10 CFR 100.21(c)(2), and GDC 19. The staff finds that the applicant has provided sufficient information to adequately address COL Information Item 2.3-4.

2.3.5 Long-Term Diffusion Estimates (Related to RG 1.206, Section C.III.2, Chapter 2, C.I.2.3.5, "Long Term Atmospheric Dispersion Estimates for Routine Releases"

2.3.5.1 *Introduction*

The long-term diffusion estimates are used to determine the amount of airborne radioactive materials expected to reach a specific location during normal operations. The diffusion estimates address the requirement concerning atmospheric dispersion and dry deposition estimates for routine releases of radiological effluents to the atmosphere. The review covers the following specific areas: (1) atmospheric dispersion and deposition models used to calculate concentrations in air and amount of material deposited as a result of routine releases of radioactive material to the atmosphere; (2) meteorological data and other assumptions used as input to the atmospheric dispersion models; (3) derivation of diffusion parameters (e.g., σ_z); (4) atmospheric dispersion (relative concentration) factors (χ/Q values) and deposition factors (D/Q values) used for assessment of consequences of routine airborne radioactive releases; (5) points of routine release of radioactive material to the atmosphere, the characteristics of each release mode, and the location of potential receptors for dose computations; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.3.5.2 *Summary of Application*

Section 2.3 of the LNP COL FSAR, Revision 9 incorporates by reference Section 2.3 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.3, the applicant provided the following:

AP1000 COL Information Item

- LNP COL 2.3-5

The applicant provided additional information in LNP COL 2.3-5 to address COL Information Item 2.3-5 (COL Action Item 2.3.5-1). LNP COL 2.3-5 addresses long-term χ/Q and D/Q estimates for calculating concentrations in air and the amount of material deposited on the ground as a result of routine releases of radiological effluents to the atmosphere during normal plant operation.

In addition, this LNP COL FSAR section addresses Interface Item 2.4 related to the limiting meteorological parameters (χ/Q values) for routine releases.

2.3.5.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for long-term diffusion estimates are given in Section 2.3.5 of NUREG-0800.

The applicable regulatory requirements for the applicant's description of atmospheric dispersion and dry deposition estimates for routine releases of radiological effluents to the atmosphere are as follows:

- 10 CFR Part 20, Subpart D, with respect to demonstrating compliances with dose limits for individual members of the public.

- 10 CFR 50.34a, "Design objectives for equipment to control releases of radioactive material in effluents—nuclear power reactors," and Sections II.B, II.C, and II.D of Appendix I of 10 CFR Part 50, with respect to the numerical guides for design objectives and limiting conditions for operation to meet the requirements that radioactive material in effluents released to unrestricted area be kept ALARA.

- 10 CFR 100.21(c)(1) with respect to establishing atmospheric dispersion site characteristics such that radiological effluent release limits associated with normal operation can be met for any individual located offsite.

The following RGs are applicable to this section:

- RG 1.23, "Meteorological Monitoring Programs for Nuclear Power Plants," Revision 1

- RG 1.109, "Calculation of Annual Doses to Man from Routine Releases of Reactor Effluents for the Purpose of Evaluating Compliance with 10 CFR Part 50, Appendix I," Revision 1

- RG 1.111, "Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors," Revision 1

- RG 1.112, "Calculation of Releases of Radioactive Materials in Gaseous and Liquid Effluents from Light-Water-Cooled Power Reactors," Revision 1

The related acceptance criteria from Section 2.3.5 of NUREG-0800 are as follows:

- A detailed description of the atmospheric dispersion and deposition models used by the applicant to calculate annual average concentrations in air and amount of material deposited as a result of routine releases or radioactive materials to the atmosphere.

- A discussion of atmospheric diffusion parameters, such as vertical plume spread (σ_z) as a function of distance, topography, and atmospheric conditions.

- Meteorological data summaries (onsite and regional) used as input to the dispersion and deposition models.

- Points of routine release of radioactive material to the atmosphere, including the characteristics (e.g., location, release mode) of each release point.

- The specific location of potential receptors of interest (e.g., nearest vegetable garden, nearest resident, nearest milk animal, and nearest meat cow in each 22½ degree direction sector within a 5-mi [8-km] radius of the site).

- The χ/Q and D/Q values to be used for assessment of the consequences of routine airborne radiological releases as described in Section 2.3.5.2 of RG 1.206: (1) maximum annual average χ/Q values and D/Q values at or beyond the site boundary and at specified locations of potential receptors of interest utilizing appropriate meteorological data for each routine venting location; and (2) estimates of annual average χ/Q values and D/Q values for 16 radial sectors to a distance of 50 mi (80 km) from the plant using appropriate meteorological data.

2.3.5.4 *Technical Evaluation*

The NRC staff reviewed Section 2.3.5 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the long-term diffusion estimates. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff reviewed the information in the LNP COL FSAR:

AP1000 COL Information Item

- LNP COL 2.3-5

The NRC staff reviewed LNP COL 2.3-5 related to the long-term diffusion estimates included in Section 2.3.5 of the LNP COL FSAR. COL Information Item 2.3-5 (COL Action Item 2.3.5-1) in Section 2.3.6.5 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will address long-term diffusion estimates and χ/Q values specified in subsection 2.3.5. The Combined License applicant should consider topographical characteristics in the vicinity of the site for restrictions of horizontal and/or vertical plume spread, channeling or other changes in airflow trajectories, and other unusual conditions affecting atmospheric transport and diffusion between the source and receptors. No further action is required for sites within the bounds of the site parameter for atmospheric dispersion.
>
> With regard to environmental assessment, the COL applicant will also provide estimates of annual average χ/Q values for 16 radial sectors to a distance of 50 mi from the plant.

2.3.5.4.1 Atmospheric Dispersion Model

The applicant used the NRC-sponsored computer code XOQDOQ (described in NUREG/CR-2919, "XOQDOQ Computer Program for the Meteorological Evaluation of Routine Releases at Nuclear Power Stations") to estimate χ/Q and D/Q values resulting from routine releases. The XOQDOQ model implements the constant mean wind direction methodology outlined in RG 1.111, "Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors," Revision 1.

The XOQDOQ model is a straight-line Gaussian plume model based on the theoretical assumption that material released to the atmosphere will be normally distributed (Gaussian) about the plume centerline. In predictions of χ/Q and D/Q values for long time periods (i.e., annual averages), the plumes horizontal distribution is assumed to be evenly distributed within the downwind direction sector (e.g., "sector averaging"). A straight-line trajectory is assumed between the release point and all receptors.

2.3.5.4.2 Release Characteristics and Receptors

The applicant modeled one ground-level release point, assuming a minimum building cross-sectional area of 2,730 m^2 and a building height of 43.9 m (based on AP1000 DCD Figure 3.8.2-1), which is smaller than the height of the entire containment building at 71.4 m. This difference of height is acceptable to the staff because the applicant's building height directly leads to assuming a smaller building cross-section. This is a conservative assumption because a smaller building cross-section will lead to less air turbulence and higher χ/Q values.

The applicant assumed a ground-level release to model routine releases. A ground-level release is a conservative assumption at a relatively flat terrain site such as LNP, resulting in higher χ/Q and D/Q values when compared to a mixed-mode (e.g., part-time ground, part-time elevated) release or a 100-percent elevated release, as discussed in RG 1.111, Revision 1. A ground-level release assumption is, therefore, acceptable to the staff.

The distance to the receptors of interest (i.e., milk cow, milk goat, garden, meat animal, and resident) were presented in LNP COL FSAR Table 2.3.5-201. For sectors not containing a certain receptor type, the applicant assumed a distance of 5 mi. The applicant calculated the distances to each of these receptors from a location defined as the mid-point of the two proposed units. However, the staff has determined that using a shorter distance (outer edge of the power block area) results in χ/Q and D/Q values that are still bounded by the AP1000 DCD. The use of the shortest distance results in higher (more conservative) χ/Q values for ground level releases. Therefore, the assumptions presented by the applicant are acceptable to the staff.

2.3.5.4.3 Meteorological Data Input

The meteorological input to XOQDOQ used by the applicant consisted of a JFD of wind speed, wind direction, and atmospheric stability based on hourly onsite data from a 2-year period from February 1, 2007 through January 31, 2009. The wind data were obtained from the 10-m level of the onsite meteorological tower, and the stability data were derived from the vertical temperature difference (delta-temperature) measurements taken between the 60-m and 10-m levels on the onsite meteorological tower.

Based on the applicant's responses to all RAIs related to the acceptability of the hourly meteorological data as discussed in SER Section 2.3.3, the staff considers the February 1, 2007, through January 31, 2009, onsite meteorological database suitable for input to the XOQDOQ model.

In response to RAI 2.3.5-3, the applicant stated that the proximity of the Gulf of Mexico to the site would be expected to have an influence on the wind direction and wind speed measurements. This influence would be expected to result in higher predictions of relative concentration (χ/Q) and relative deposition (D/Q), due to either low wind speeds or to an increase in the frequency of wind directions in specific sectors. This would be a result of the sea-breeze and land-breeze circulations expected near coastal sites. This pattern can be identified in LNP COL FSAR Tables 2.3.5-201 through 2.3.5-204. These tables show that the highest χ/Q and D/Q values are found in the WSW and W quadrants. This pattern corresponds with the downwind sectors of the most frequent wind directions identified in LNP COL FSAR Figure 2.3.2-201. The NRC was able to confirm the applicant's discussion through analysis of the hourly onsite data collected during a 2-year period from February 1, 2007 through January 31, 2009. Therefore, the staff finds the applicant's response acceptable and considers RAI 2.3.5-3 closed.

2.3.5.4.4 Diffusion Parameters

The applicant chose to implement the diffusion parameter assumptions outlined in RG 1.111, Revision 1, as a function of atmospheric stability, for its XOQDOQ model runs. The staff evaluated the applicability of the XOQDOQ diffusion parameters and concluded that no unique topographic features preclude the use of the XOQDOQ model for the LNP site. Therefore, the staff finds that the applicant's use of diffusion parameter assumptions, as outlined in RG 1.111, Revision 1 was acceptable.

In response to RAI 2.3.5-6, the applicant provided justification for the use of the XOQDOQ straight-line model trajectory model. The applicant stated that the LNP site is located in an area that is surrounded by flat terrain for a distance of more than 50 mi and that no significant special variations in dispersion or direction are expected to occur as a result of variations in terrain. The staff has reviewed this RAI response and agrees with the qualitative and quantitative statements made by the applicant. Therefore, the staff finds the applicant's response acceptable and considers RAI 2.3.5-6 closed.

2.3.5.4.5 Resulting Relative Concentration and Relative Deposition Factors

LNP COL FSAR Tables 2.3.5-201 through 2.3.5-204 lists the long-term atmospheric dispersion and deposition estimates for the EAB, LPZ, and special receptors of interest that the applicant derived from their XOQDOQ modeling results. LNP COL FSAR Tables 2.3.5-201 through 2.3.5-204 also describe the applicant's long-term atmospheric dispersion and deposition estimates for 16 radial sectors from the site boundary to a distance of 50-mi from the proposed facility.

The χ/Q values presented in LNP COL FSAR Tables 2.3.5-201 through 2.3.5-204 reflect several plume radioactive decay and deposition estimates for the EAB, LPZ, and special receptors of interest that the applicant derived from its XOQDOQ modeling results.

The χ/Q values presented in LNP COL FSAR Tables 2.3.5-201 through 2.3.5-204 reflect several plume radioactive decay and deposition scenarios. Section C.3 of RG 1.111, Revision 1 states that radioactive decay and dry deposition should be considered in radiological impact evaluations of potential annual radiation doses to the public, resulting from routine releases of radioactive materials in gaseous effluents. Section C.3.a of RG 1.111, Revision 1 states that an overall half-life of 2.26 days is acceptable for evaluating the radioactive decay of short-lived noble gases and an overall half-life of 8-days is acceptable for evaluating the radioactive decay for all iodines released to the atmosphere. Definitions for the χ/Q categories are as follows:

- Undepleted/No Decay χ/Q values are χ/Q values used to evaluate ground-level concentrations of long-lived noble gases, tritium, and carbon-14. The plume is assumed to travel downwind, without undergoing dry deposition of radioactive decay

- Undepleted/2.26-Day Decay χ/Q values are χ/Q values used to evaluate ground-level concentrations of short-lived noble gases. The plume is assumed to travel downwind, without undergoing dry deposition, but is decayed, assuming a half-life of 2.26 days, based on the half-life of xenon-133.

- Depleted/8.00-Day Decay χ/Q values are χ/Q values used to evaluate ground-level concentrations of radioiodine and particulates. The plume is assumed to travel downwind, with dry deposition, and is decayed assuming a half-life of 8.00 days, based on the half-life of iodine-131.

Using the information provided by the applicant, including the 10-m level JFDs of wind speed, wind direction, and atmospheric stability presented in LNP COL FSAR Tables 2.3.2-201 through 2.3.2-208, the staff confirmed the applicant's χ/Q and D/Q values by running the XOQDOQ computer code and obtaining similar results (i.e., values on average within about 1-percent). The JFDs used by the applicant for the long-term diffusion estimates consisted of 11 wind speed categories. These wind speed categories were based on RG 1.23, Revision 1, but combined the first two non-calm wind speed categories into one category of 1.0-1.05 m/s. In light of the foregoing, the staff accepts the long-term χ/Q and D/Q values presented by the applicant.

COL Information Item 2.3-5 also states that, with regard to environmental assessment, estimates of annual average χ/Q values for 16 radial sectors to a distance of 50-mi from the plant should be provided. The applicant provided these values in LNP COL FSAR Tables 2.3.5-201 through 2.3.5-204. Using staff generated JFDs and the XOQDOQ computer code, these χ/Q values were confirmed by the staff and were found to be adequate and acceptable.

2.3.5.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.3.5.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to long-term diffusion estimates and there is no outstanding information expected to be addressed in the LNP COL FSAR relating to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

COL Information Item 2.3-5 states that a COL applicant shall address the site-specific diffusion estimates and χ/Q values as specified in AP1000 DCD Section 2.3.5. Based on the meteorological data provided by the applicant and an atmospheric dispersion model that is appropriate for the characteristics of the site and release points, the staff concludes that representative atmospheric dispersion and deposition factors have been calculated for 16 radial sectors from the site boundary to a distance of 50-mi (80-km) as well as for specific locations of potential receptors of interest. The characterization of atmospheric dispersion and deposition conditions are acceptable to meet the criteria described in RG 1.111, Revision 1 and are appropriate for the evaluation to demonstrate compliance with the numerical guides for doses in Subpart D of 10 CFR Part 20 and Appendix I to 10 CFR Part 50. The staff finds that the applicant has provided sufficient information to adequately address COL Information Item 2.3-5.

2.4 Hydrologic Engineering

To ensure that one or more nuclear power plants can be safely operated on the applicant's proposed site and in accordance with the Commission's regulations, NRC staff evaluated the hydrologic site characteristics of the proposed site. These site characteristics included the maximum flood elevation of surface water and the maximum elevation of groundwater. The staff also described the characteristic ability of the site to attenuate a postulated accidental release of radiological material into surface water and groundwater before it reaches a receptor.

The staff prepared Sections 2.4.1 through 2.4.14 of this SER in accordance with the review procedures described in NUREG-0800, "Standard Review Plan [SRP] for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," using information presented in Section 2.4, "Hydrologic Engineering," of the Progress Energy Florida[9] (PEF) LNP Units 1 and 2 FSAR Revision 4, DCD Revision 19, applicant's responses to staff RAIs, and generally available reference materials (e.g., those cited in applicable sections of NUREG-0800).

The ultimate heat sink of the AP1000 reactor is the atmosphere. Therefore, hydrologic characteristics associated with conditions that would result in a loss of external water supply (e.g., low water, channel diversions) are not relevant for this particular design. Also, seismic design considerations of water supply structures are not relevant for this particular design. Therefore, Regulatory Guide (RG) 1.27, "Ultimate Heat Sink for Nuclear Power Plants" and RG 1.29, "Seismic Design Classification" were not a necessary part of the regulatory basis for this Section 2.4 review.

In Part 7 of the Combined License Application, the applicant described an administrative departure (STD DEP 1.1-1) that remaps Section 2.4 section numbers to the associated DCD section numbers. The staff determines that this departure has no safety significance.

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[9] The applicant, Duke Energy Florida, LLC, was formerly identified as Progress Energy Florida, Inc. and Duke Energy Florida, Inc. In a letter dated April 15, 2013, Progress Energy Florida notified the NRC that its name was changing to Duke Energy Florida effective April 29, 2013. The name changes and a 2012 corporate merger between Duke Energy and Progress Energy are described in Chapter 1 of the SER. Because a portion of the review described in this chapter was completed prior to the name change, the NRC staff did not change references to "Progress Energy Florida" or "PEF" to "Duke Energy Florida" or "DEF" in this chapter.

2.4.1 Hydrologic Description

2.4.1.1 *Introduction*

FSAR Section 2.4.1 of the LNP COL application described the site and all safety-related elevations, structures and systems from the standpoint of hydrologic considerations and provided a topographic map showing the proposed changes to grading and to natural drainage features.

Section 2.4.1 of this SER provides a review of the following specific areas: (1) interface of the plant with the hydrosphere including descriptions of site location, major hydrologic features in the site vicinity, surface water and groundwater characteristics, and the proposed water supply to the plant; (2) hydrologic causal mechanisms that may require special plant design bases or operating limitations with regard to floods and water supply requirements; (3) current and likely future surface and groundwater uses by the plant and water users in the vicinity of the site that may affect the safety of the plant; (4) available spatial and temporal data relevant for the site review; (5) alternate conceptual models of the hydrology of the site that reasonably bound hydrologic conditions at the site; (6) potential effects of seismic and non-seismic data on the postulated design bases and how they relate to the hydrology in the vicinity of the site and the site region; and (7) any additional information requirements prescribed within the “Contents of Application” sections of the applicable Subparts to 10 CFR Part 52.

As stated in Section 2.4 above, hydrologic characteristics associated with conditions that would result in a loss of external water supply and seismic design considerations of water supply structures are not relevant for the AP1000 design. Therefore, item (6) above was not part of the staff’s review.

2.4.1.2 *Summary of Application*

This section of the LNP COL FSAR describes the site and all safety-related elevations, structures and systems from the standpoint of hydrologic considerations and provided a topographic map showing the proposed changes to grading and to natural drainage features. The applicant addressed these issues as follows:

<u>*COL Information Item*</u>

- LNP COL 2.4-1 Hydrological Description

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.1 of Revision 19 of the DCD.

> Combined License applicants referencing the AP1000 certified design will describe major hydrologic features on or in the vicinity of the site including critical elevations of the nuclear island and access routes to the plant.

2.4.1.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification of floods and flood design considerations, and the associated acceptance criteria, are described in Section 2.4.1 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying site location and description of the site hydrosphere are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site.

- 10 CFR 100.20(c), regarding requirements to consider physical site characteristics in site evaluations.

- 10 CFR 52.79(a)(1)(iii), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The related acceptance criteria are as follows:

- RG 1.59, "Design Basis Floods for Nuclear Power Plants" (NRC 1977a), as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants" (NRC 1976b).

2.4.1.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.1 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[10] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the site hydrological description. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

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[10] See Section 1.2.2 for a discussion of the staff's review related to verification of the scope of information to be included in a COL application that references a DC.

2.4.1.4.1 Site and Facilities

<u>Information Submitted by the Applicant</u>

The LNP site, 1,257 ha (3,105 ac) in size, is located southwest of Gainesville and west of Ocala in southern Levy County in Florida (Figure 2.4.1-1), approximately 12.8 km (8 mi) inland from the Gulf of Mexico, 4.8 km (3 mi) north of Lake Rousseau, and 15.5 km (9.6 mi) north of PEF's Crystal River Energy Complex (CREC). The two proposed units will be called LNP Unit 1 and LNP Unit 2.

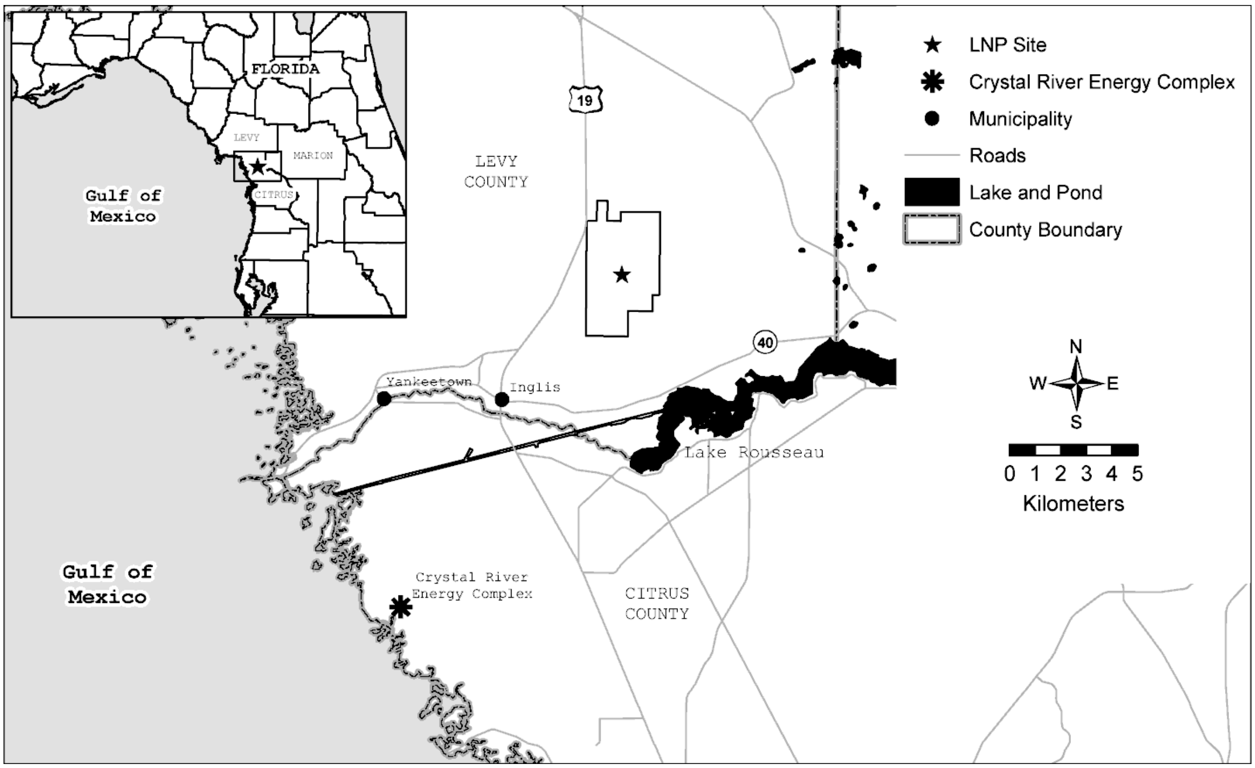


Figure 2.4.1-1. The LNP Site and Surrounding Area

Elevations at the LNP site range from 9.1 to 18.3 m (30 to 60 ft) National Geodetic Vertical Datum of 1929 (NGVD29). The applicant stated that the nominal plant grade would be 15.5 m (51 ft) NAVD88 with actual plant grade lower than 15.5 m (51 ft) NAVD88 (North American Vertical Datum of 1988) to accommodate drainage for local flooding. At the site audit, the applicant stated that elevation values referring to NGVD29 are approximately 0.3 m (1 ft) higher than the corresponding NAVD88 value on an average for the LNP site.

The Gulf of Mexico, the Cross Florida Barge Canal (CFBC), the Withlacoochee River, and Lake Rousseau are the major hydrologic features located near the LNP site. A 13.4-km (8.3 mi) stretch of the CFBC runs from below the Inglis Dam that impounds Lake Rousseau on the

Withlacoochee River to the Gulf of Mexico. Inglis Lock, Inglis Bypass Channel and Spillway, and the Inglis Dam are three water-control structures in the LNP site area and are operated by the South West Florida Water Management District (SWFWMD).

As stated in the FSAR, the proposed units will use a closed-loop normal cooling system with mechanical draft cooling towers. A new intake on the CFBC will provide cooling water for normal plant cooling. Two pipelines, one for each LNP unit, will discharge blowdown from the cooling towers to the existing CREC discharge canal. Onsite wells will provide water needed for general plant operations, including makeup to the service water system, potable water supply, and raw water to demineralized water, fire protection water, and media filter backwash.

NRC Staff's Technical Evaluation

The staff reviewed the information provided by the applicant to determine the adequacy of the information in support of hydrologic site characterization for the purpose of siting a nuclear reactor. The specific hydrology-related site characterization of the LNP site with respect to general description of the hydrosphere as described in NUREG-0800 (NRC 2007a) includes local intense precipitation, site drainage, probable maximum flood and associated water surface elevations, dam breaches and resulting flood elevations, storm surges and seiches with related flooding and low-water effects, tsunamis and associated flooding, ice formation, channel diversion, flooding protection requirements, safety-related water use, groundwater elevations, and accidental release of liquid radioactive effluents to ground and surface waters. The staff used the location of the LNP site, its hydrological and meteorological characteristics, and the interface of the plant with the elements of the hydrosphere to determine the site characteristics for safe siting and operation of the proposed LNP Units 1 and 2.

To ascertain the safe operation of a reactor at a site, the staff requires an accurate description of the site, the site region, and facilities at the site, including all safety-related facilities to determine whether the most conservative of plausible conceptual models are identified. In RAI 2.4.1-1, the staff requested additional information regarding the applicant's process to determine the conceptual models of the interface of the plant with the hydrosphere, including the hydrologic causal mechanisms to ensure that the most conservative of plausible conceptual models have been identified. In a letter dated June 15, 2009 (ML091680037), the applicant stated that the LNP site was characterized using conceptual modes that describe flooding from local intense precipitation, flooding in rivers and streams, flooding from upstream dam failures, and flooding from surges and tsunamis. In addition, the applicant also used conceptual site models to characterize subsurface properties and the accidental release of radioactive liquids.

The applicant stated that published information from local, State, and Federal agencies was used to document the physiography, hydrology, geology, meteorology, topography, and demography near the LNP site. The applicant also collected geological, hydrogeological, meteorological, and water quality data near the LNP site. The aforementioned data and information were used to develop site conceptual models. The applicant stated that conceptual site models developed for individual flood mechanisms, subsurface characteristics, and surface and subsurface pathways are described in responses to the staff's RAI corresponding to the respective FSAR sections.

The staff reviewed the applicant's response to RAI 02.04.01-01 and determined that the applicant appropriately used information and data published by local, State, and Federal agencies in addition to site-specific data to conceptualize the hydrologic mechanisms and site characteristics that may affect safety of proposed LNP Units 1 and 2. The staff concluded, therefore, that the applicant has provided sufficient information for describing the interface of the plant with the hydrosphere and to characterize the hydrologic causal mechanisms at and near the LNP site.

To perform its safety assessment, the staff requires an accurate description of the site, the site region, and facilities at the site, including all safety-related facilities. The staff conducted a hydrology site audit November 4–6, 2008. The staff's audit included a tour of the LNP site, the meteorological tower, the CFBC, the proposed makeup water intake location, the Inglis Lock, and the Inglis Bypass Channel and Spillway. To determine the accuracy and acceptability of the models used to estimate the design-basis flood, the staff issued **RAI 02.04.01-02**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17 and 10 CFR Part 100, the applicant should include a complete description of all spatial and temporal datasets used in support of its conclusions regarding safety of the plant. Data and descriptions should be sufficiently detailed to allow the staff to review the applicant's conclusions regarding the safety of the plant and to determine ~~of~~ the design bases of safety-related SSCs. Please provide input and output files associated with the HEC-HMS and HEC-RAS model simulations performed for the FSAR.

The applicant responded to the staff's RAI 02.04.01-02 in a letter dated June 23, 2009 (ML091830343). The applicant provided U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS; USACE 2010a) input and sample output data sets along with model control specifications and meteorological data. The applicant also provided USACE Hydrologic Engineering Center-River Analysis System (HEC-RAS; USACE 2010b) input and sample output datasets along with geometry data.

The staff reviewed the data sets provided by the applicant and determined that these data sets were suitable for staff to independently carry out a review of the applicant's flooding analyses. Subsequent subsections of this report describe the staff's independent and confirmatory analyses to verify the applicant's safety conclusions. To determine the appropriate and consistent usage of datums and elevations, the staff issued **RAI 02.04.01-03**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17 and 10 CFR Part 100, the applicant should include a complete description of all spatial and temporal datasets used by the applicant in support of its conclusions regarding safety of the plant. Data and descriptions should be sufficiently detailed to allow the staff to review the applicant's conclusions regarding the safety of the plant and to determine the design bases of safety-related SSCs. Please provide clarification regarding the use of the term MSL in the FSAR and clearly state the units of measurements and the contour interval on all the pertinent figures in the FSAR.

The applicant responded to the staff's RAI 02.04.01-03 in a letter dated June 15, 2009 (ML091680037). The applicant confirmed that its use of the term MSL in the FSAR can be converted to NGVD29 (Elevation ft NGVD29 = X ft MSL - 0.893 ft.). The applicant identified locations in the FSAR and changed text to replace the term MSL (or msl) with NGVD29. The applicant also stated the approximate elevation offset to convert elevations expressed in NGVD29 to NAVD88. The applicant also identified and fixed a typographical error. The applicant appropriately annotated some FSAR figures. The applicant made these changes in FSAR Revision 4.

The staff reviewed the applicant's response and determined that the applicant has corrected the inconsistencies in the FSAR. The staff independently used the National Oceanic and Atmospheric Administration (NOAA) National Geodetic Survey (NGS) VERTCON tool (NGS 2011) to verify that elevations near the LNP site referring to the NGVD29 datum are 0.31 m (1 ft) greater than those referring to the NAVD88 datum. Based on its independent review, the staff determined that the applicant's response to RAI 02.04.01-03 is acceptable.
The staff compared the information presented by the applicant in FSAR Section 2.4.1 with publicly available maps and data regarding the LNP site and its surrounding region. The proposed LNP site is located in Florida's Levy County approximately 71 km (44 mi) south-southwest from the City of Gainesville, Florida; 8 km (5 mi) east-northeast of Yankeetown, Florida; 4.8 km (3 mi) north of Inglis Lock on Lake Rousseau; and 16 km (10 mi) northeast of the CREC (Figure 2.4.1-1). The Gulf of Mexico is located approximately 13.7 km (8.5 mi) west-southwest of the LNP site.

2.4.1.4.2 Hydrosphere

<u>*Information Submitted by the Applicant*</u>

The LNP site lies mainly in the Waccasassa River Basin, with a small portion falling in the Withlacoochee River Basin (Figure 2.4.1-2). There are no named streams on the LNP site and the drainage is mainly overland toward the Lower Withlacoochee River and the Gulf of Mexico located southwest of the LNP site. Freshwater bodies in the vicinity include the Withlacoochee River and Lake Rousseau. Wetlands dominate the LNP site. Salt marshes are located between Highway 19 located west of the site and the Gulf of Mexico.

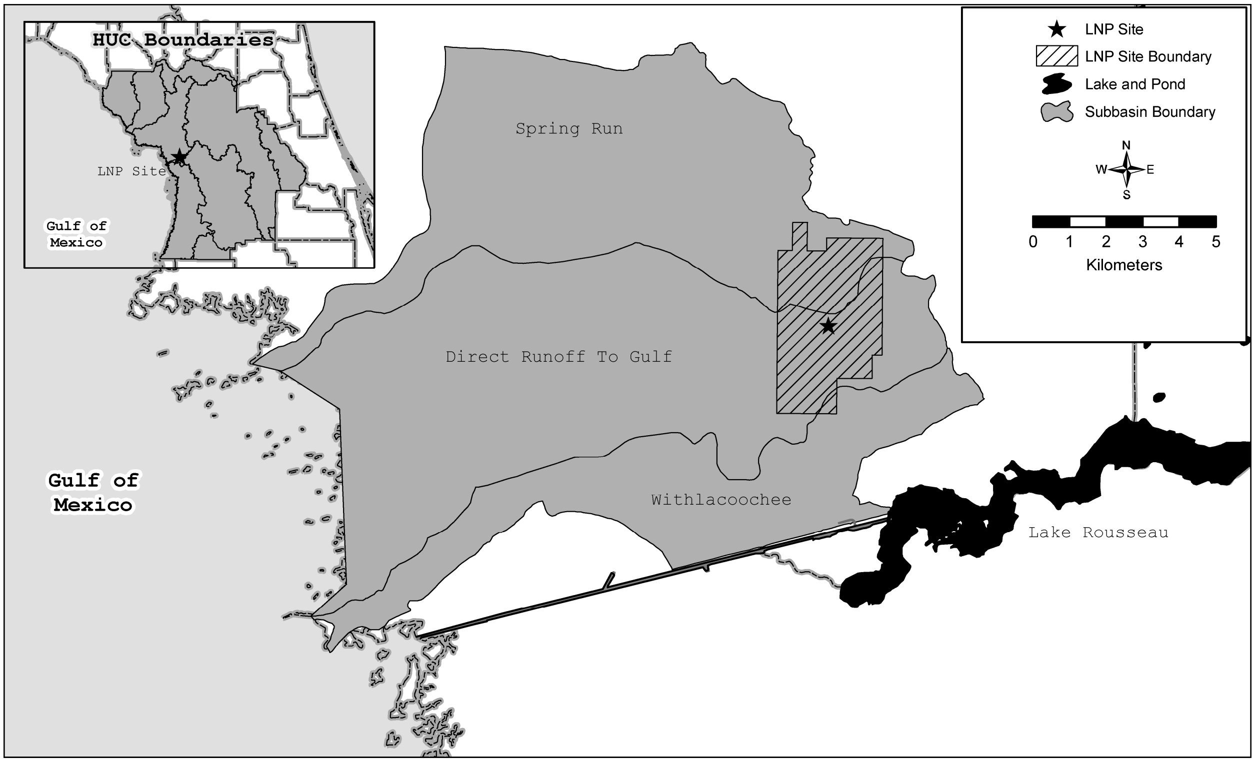


Figure 2.4.1-2. The Subbasins Within Which the LNP Site is Located

The Withlacoochee River Basin which has an area of 14,087 km^2 (5,439 mi^2), is partially located in the northern portion of the SWFWMD. The Withlacoochee River originates in Green Swamp and flows northwest approximately 253 km (157 mi) before discharging into the Gulf of Mexico near Yankeetown (Figure 2.4.1-3). The average gradient of the river is approximately 0.17 m/km (0.9 ft/mi). Little Withlacoochee River, Big Grant Canal, Jumper Creek, Shady Brook, Outlet River of Lake Panasoffkee, Leslie Heifner Canal, Orange State Canal, Tsala Apopka Outfall Canal, and Rainbow River are the major tributaries of the river. The Withlacoochee River and the Rainbow River contribute most of the water to Lake Rousseau.

The Upper Withlacoochee River extends from its headwaters in Green Swamp to its confluence with the Little Withlacoochee River. The Middle Withlacoochee River extends from its confluence with the Little Withlacoochee River downstream to U.S. Highway 41 approximately 1.0 km (0.6 mi) east of Lake Rousseau. The Lower Withlacoochee River extends from U.S. Highway 41 to its discharge in the Gulf of Mexico and includes Lake Rousseau, a portion of the CFBC, and the three water-control structures mentioned above. Rainbow River, fed by a first order natural spring, is 9.2 km (5.7 mi) in length and discharges approximately 21 m^3/s (727 cfs) daily into the Withlacoochee River.

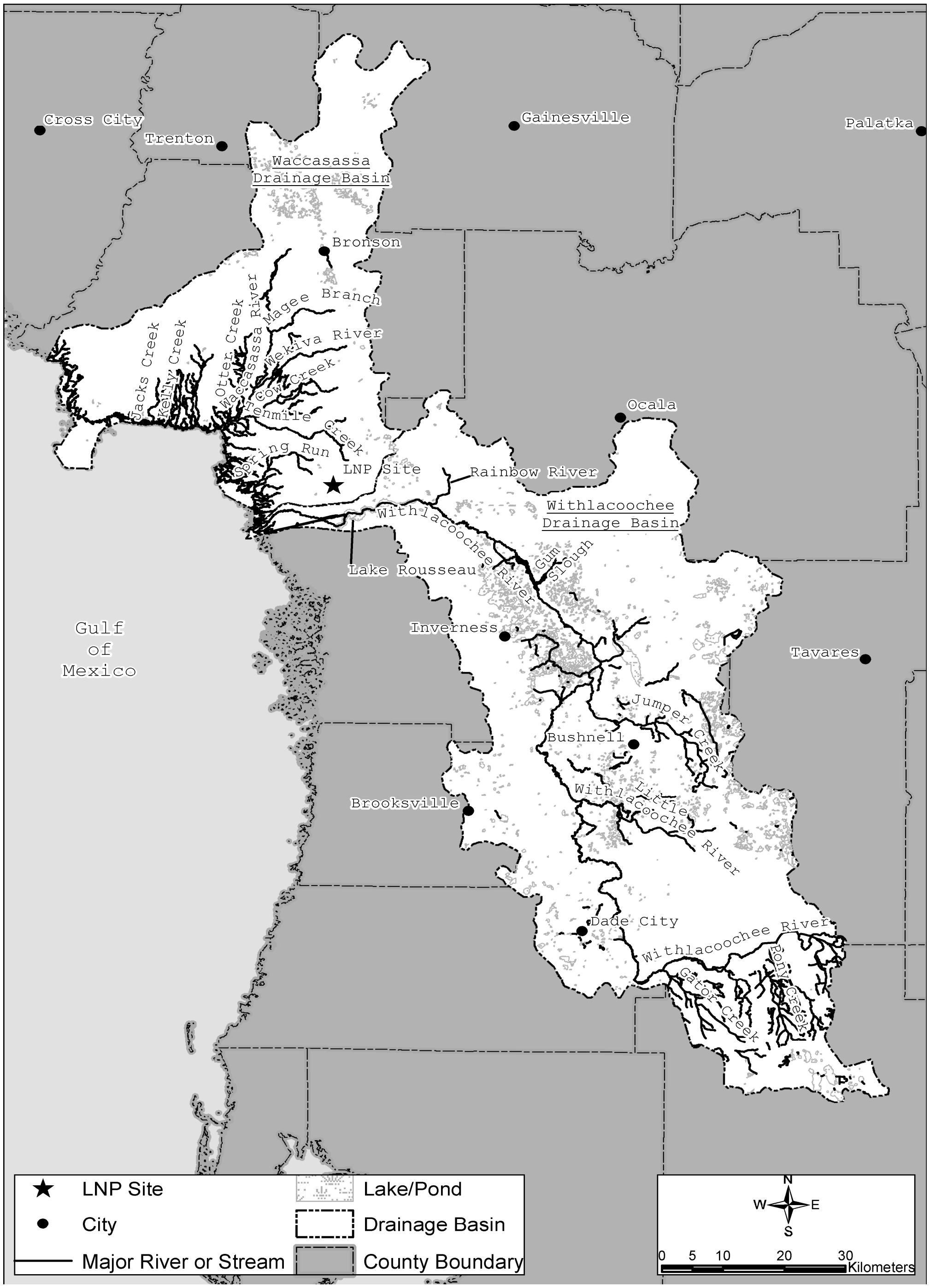
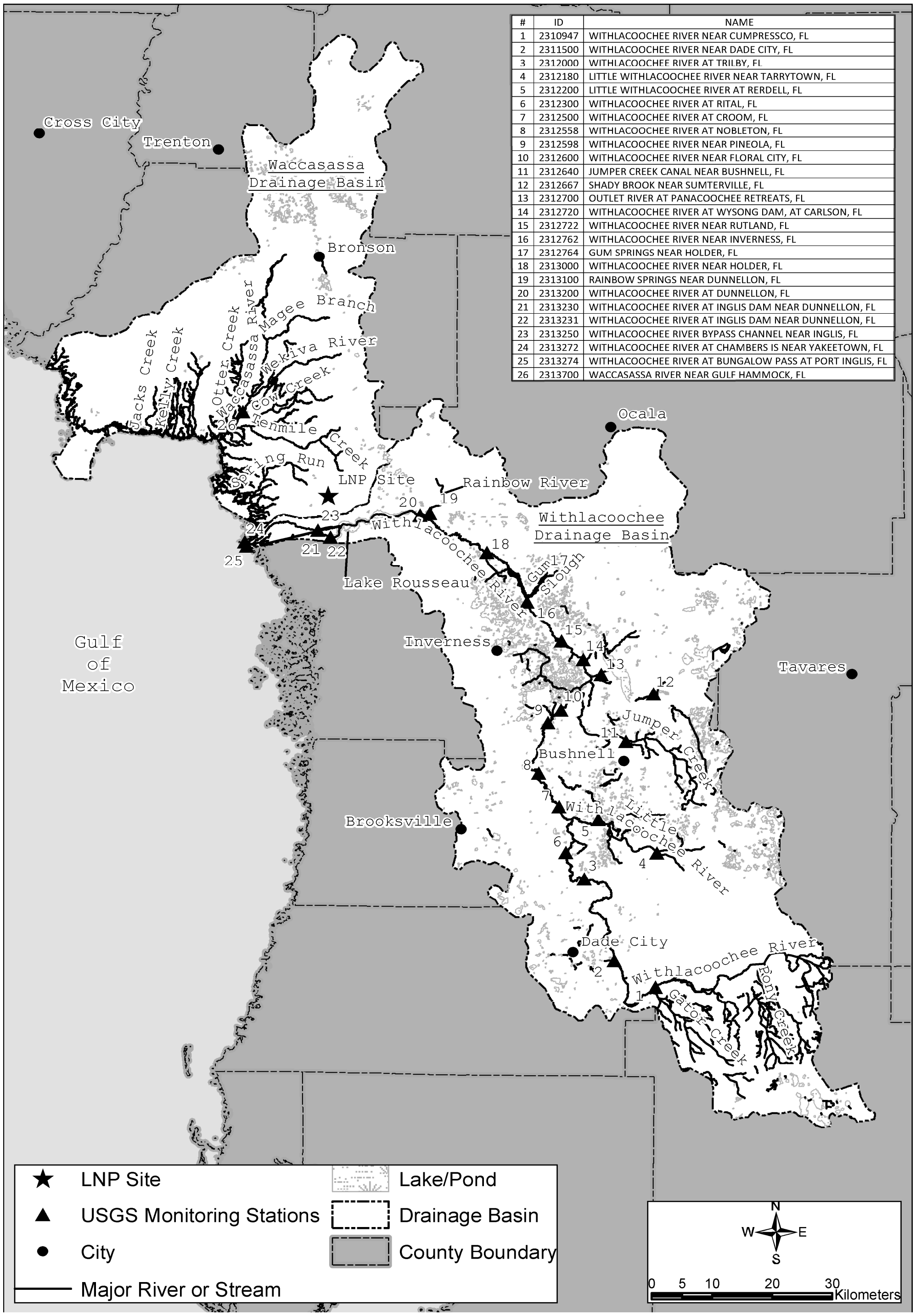


Figure 2.4.1-3. The Withlacoochee and Waccasassa River Basins



#	ID	NAME
1	2310947	WITHLACOOCHEE RIVER NEAR CUMPRESSCO, FL
2	2311500	WITHLACOOCHEE RIVER NEAR DADE CITY, FL
3	2312000	WITHLACOOCHEE RIVER AT TRILBY, FL
4	2312180	LITTLE WITHLACOOCHEE RIVER NEAR TARRYTOWN, FL
5	2312200	LITTLE WITHLACOOCHEE RIVER AT RERDELL, FL
6	2312300	WITHLACOOCHEE RIVER AT RITAL, FL
7	2312500	WITHLACOOCHEE RIVER AT CROOM, FL
8	2312558	WITHLACOOCHEE RIVER AT NOBLETON, FL
9	2312598	WITHLACOOCHEE RIVER NEAR PINEOLA, FL
10	2312600	WITHLACOOCHEE RIVER NEAR FLORAL CITY, FL
11	2312640	JUMPER CREEK CANAL NEAR BUSHNELL, FL
12	2312667	SHADY BROOK NEAR SUMTERVILLE, FL
13	2312700	OUTLET RIVER AT PANACOOCHEE RETREATS, FL
14	2312720	WITHLACOOCHEE RIVER AT WYSONG DAM, AT CARLSON, FL
15	2312722	WITHLACOOCHEE RIVER NEAR RUTLAND, FL
16	2312762	WITHLACOOCHEE RIVER NEAR INVERNESS, FL
17	2312764	GUM SPRINGS NEAR HOLDER, FL
18	2313000	WITHLACOOCHEE RIVER NEAR HOLDER, FL
19	2313100	RAINBOW SPRINGS NEAR DUNNELLON, FL
20	2313200	WITHLACOOCHEE RIVER AT DUNNELLON, FL
21	2313230	WITHLACOOCHEE RIVER AT INGLIS DAM NEAR DUNNELLON, FL
22	2313231	WITHLACOOCHEE RIVER AT INGLIS DAM NEAR DUNNELLON, FL
23	2313250	WITHLACOOCHEE RIVER BYPASS CHANNEL NEAR INGLIS, FL
24	2313272	WITHLACOOCHEE RIVER AT CHAMBERS IS NEAR YAKEETOWN, FL
25	2313274	WITHLACOOCHEE RIVER AT BUNGALOW PASS AT PORT INGLIS, FL
26	2313700	WACCASASSA RIVER NEAR GULF HAMMOCK, FL

Figure 2.4.1-4. USGS Streamflow Gauges in the Withlacoochee and the Waccasassa River Basins

Figure 2.4.1-4 shows six USGS stream gauges near the LNP site, five on the Lower Withlacoochee River and one on the Rainbow River. At some gauges, only gauge height data are available while at other gauges both gauge height and discharge measurements are available. The applicant provided a summary of the data available at these gauges in FSAR Table 2.4.1-201.

The CFBC was conceived as a northern inland waterway between the Gulf of Mexico and northeast Florida in the 1960s. The design depth and width of the canal were 3.7 and 45.7 m (12 and 150 ft), respectively. Due to its adverse environmental and economic impact, construction of the CFBC was stopped in 1971. The CFBC bisected the original course of the Lower Withlacoochee River and severed the connection between Lake Rousseau and the original course. Water is now released from Lake Rousseau through the Inglis Bypass Channel and Spillway into the original course of the Lower Withlacoochee River. Flow through the Inglis Dam only occurs during large floods.

Lake Rousseau is a 1,685-ha (4,163-ac), 9.2-km (5.7-mi) long impoundment on the Withlacoochee River located approximately 17.7 km (11 mi) upstream of the mouth of the river near the city of Inglis. The lake was constructed in 1909 by Florida Power Corporation for power generation. The water level in the lake is controlled by the Inglis Bypass Channel and Spillway, the Inglis Dam, and the Inglis Lock. The operating level is maintained between 7.3 and 8.5 m (24 and 28 ft) NGVD29 with an optimum level at 8.4 m (27.5 ft) NGVD29. Normal discharge of 43.6 m^3/s (1,540 cfs), which is also the maximum discharge capacity of the spillway with a crest elevation of 8.5 m (28 ft) NGVD29, is passed through the Inglis Bypass Channel and Spillway. Flow exceeding this discharge is passed through the Inglis Dam to the CFBC through a short, original course of the Withlacoochee River downstream of the dam.

Inglis Lock is 182.9 m (600 ft) long and 25.6 m (84 ft) wide and was designed as a navigational lock for vessels traveling between Lake Rousseau and the Gulf of Mexico. The lock has not been used since 1999 because its upstream gate is in need of repair. There are currently no plans to repair the gate.

Inglis Dam has a reinforced concrete, two-bay, gated spillway with ogee weirs with a crest elevation of 8.5 m (28 ft) NGVD29. The maximum allowable lake level is 8.5 m (28 ft) NGVD29. Other water-control structures such as the Lake Tsala Apopka Dam, Slush Pond Dam, and Gant Lake Dam exist upstream of Lake Rousseau but do not directly affect the water level in the lake. The Tsala Apopka chain of lakes and the water-control structures are located in central portion of the Withlacoochee River Basin. The system comprises three pools: Hernando, Inverness, and Floral City. The control structures regulate flow between the river and the pools. The Floral City pool is the highest, with a high-water level of 12.7 m (41.8 ft) NGVD29 and a 10-year flood guidance level of 13.2 m (43.4 ft) NGVD29. The 10-year flood guidance levels of the Hernando and Inverness pools are 12.3 and 12.7 m (40.5 and 41.8 ft) NGVD29, respectively. The three pools range in storage capacity from 36,634,409 m^3 to 74,008,908 m^3 (29,700 to 60,000 ac-ft). The operations of the Tsala Apopka system are described by the SWFWMD (2007). The applicant stated that the USACE National Inventory of Dams lists seven dams on Saddle Creek that create settling areas. The seven Saddle Creek settling areas range in storage from 62,908 m^3 (51 ac-ft) for settling area number 6 to 19,452,008 m^3 (6 to 15,770 ac-ft) for settling area number 2. Slush Pond Dam has a storage of 62,908 m^3 (51 ac-ft) and Gant Lake Dam has a storage of 651,278 m^3 (528 ac-ft).

The relatively undeveloped Waccasassa River Basin, which has an approximate area of 2,334 km^2 (901 mi^2), is located in the southern part of the Suwannee River Water Management District. Named drainages in the basin include the Waccasassa River, Jakes Creek, Kelly Creek, Otter Creek, Magee Branch, Wekiva Creek, Cow Creek, Ten Mile Creek, and Spring Run. The basin generally drains southwest towards the Gulf of Mexico and does not have any known water-control structures.

There is no known public water supply from Lake Rousseau or from the Withlacoochee River; the primary source of public water supply is from groundwater near the LNP site.

NRC Staff's Technical Evaluation

The staff reviewed the applicant's responses to the RAIs and determined that the description of the hydrosphere and the interfaces of the proposed units with the hydrosphere are adequately accounted for in site characterization. The staff used publicly available data from USGS, Natural Resources Conservation Service (NRCS), NOAA and its own observations from the site tour to perform its review.

The staff used the Watershed Boundary Dataset available from the Natural Resources Conservation Service (NRCS 2010) to independently confirm the location of the LNP site and the hydrologic setting in its vicinity. Most of the LNP site is located in the Waccasassa River Basin in Florida. Most of the LNP site is located in subbasins named Spring Run and Thousandmile Creek-Halverson Creek Frontal (Figure 2.4.1-5). A small portion of the LNP site is located in the West Lake Rousseau-Cross Florida Barge Canal drainage, which is a subbasin of the Withlacoochee River Basin. Although Spring Run and Thousandmile Creek-Halverson Creek Frontal are subbasins of the Waccasassa River Basin, the streams within these two subbasins drain directly to the Gulf of Mexico (Figure 2.4.1-5). The West Lake Rousseau-Cross Florida Barge Canal drainage, a subbasin of the Withlacoochee River Basin, is hydrologically separate from the Waccasassa River Basin.

Based on its independent review of hydrologic data at and in the vicinity of the LNP site, the staff determined that the applicant has accurately described the hydrologic interfaces for the proposed units at the LNP site.

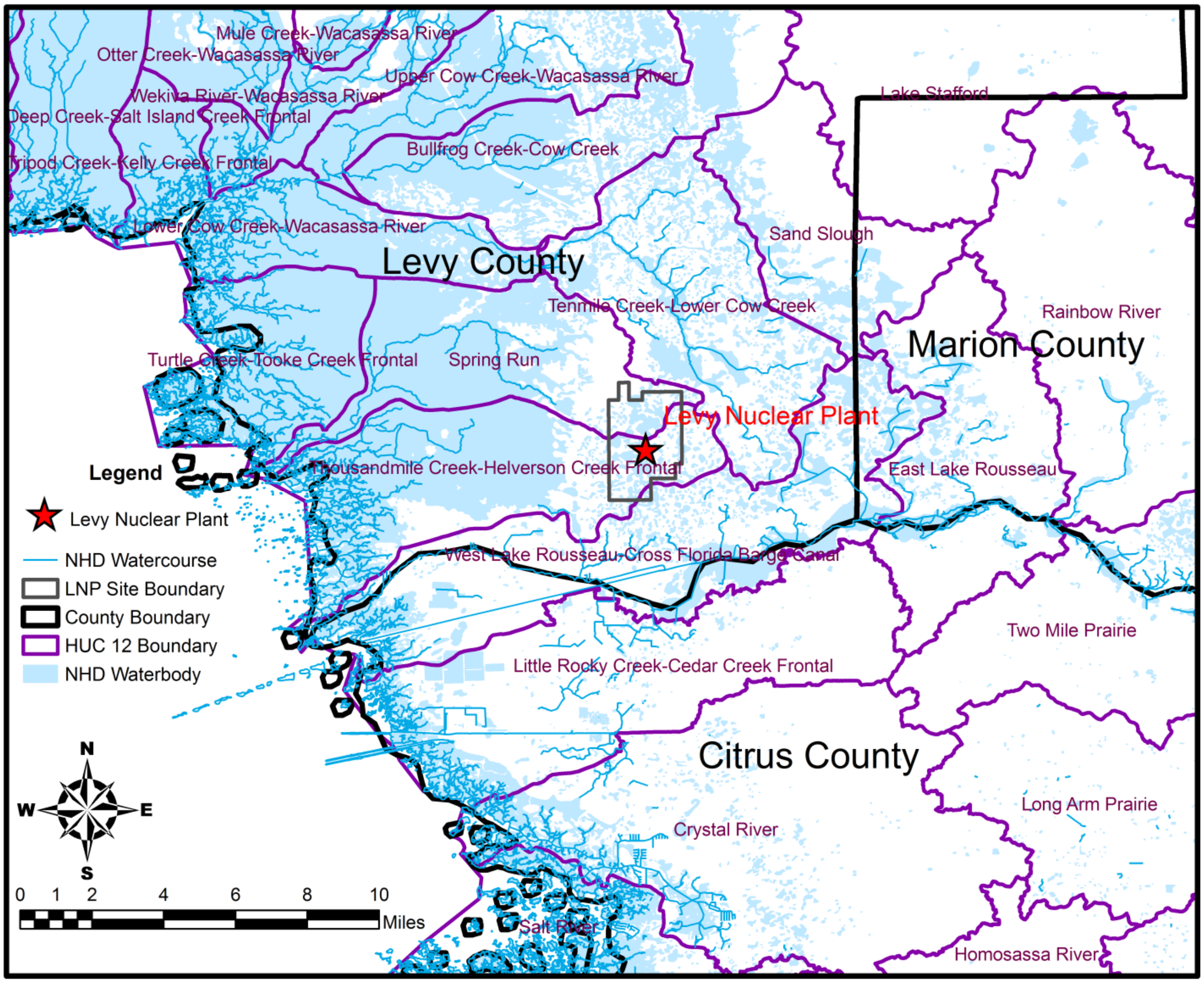


Figure 2.4.1-5. Subwatersheds Near the LNP Site. Waterbodies and watercourses data were obtained from the National Hydrography Dataset.

2.4.1.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.1.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has demonstrated that the characteristics of the site fall within the site parameters specified in the Design Certification (DC) rule, and that no outstanding information is expected to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.1 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses COL information item 2.4-1. In conclusion, the applicant has provided sufficient information for satisfying 10 CFR Part 52 and 10 CFR Part 100.

2.4.2 Floods

2.4.2.1 *Introduction*

FSAR Section 2.4.2 of the LNP COL application discusses historical flooding at the proposed site or in the region of the site. The information summarizes and identifies individual flood-producing mechanisms, and combinations of flood-producing phenomena, to establish the design-basis flood for SSCs important to safety. The discussion also covers the potential effects of local intense precipitation on SSCs important to safety.

Section 2.4.2 of this SER provides a review of the following specific areas and flood-causing mechanisms: (1) local flooding on the site and drainage design; (2) stream flooding; (3) surges; (4) seiches; (5) tsunami; (6) dam failures; (7) flooding caused by landslides; (8) effects of ice formation on waterbodies; (9) combined event criteria; (10) other site-related evaluation criteria; and (11) additional information requirements prescribed in the "Contents of Application" sections of applicable subparts to 10 CFR Part 52. Flood causing mechanisms listed above are also discussed in detail in subsequent subsections of this SER.

2.4.2.2 *Summary of Application*

This section of the COL FSAR addresses information about site-specific flooding. The applicant addressed the information as follows:

<u>*COL Information Item*</u>

- LNP COL 2.4-2

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 19 of the DCD.

> Combined License applicants referencing the AP1000 design will address the following site-specific information on historical flooding and potential flooding factors, including the effects of local intense precipitation.
>
> - Probable Maximum Flood on Streams and Rivers – Site-specific information that will be used to determine design basis flooding at the site. This information will include the probable maximum flood on streams and rivers.

- Dam Failures – Site-specific information on potential dam failures.
- Probable Maximum Surge and Seiche Flooding – Site-specific information on probable maximum surge and seiche flooding.
- Probable Maximum Tsunami Loading – Site-specific information on probable maximum tsunami loading.
- Flood Protection Requirements – Site-specific information on flood protection requirements or verification that flood protection is not required to meet the site parameter of flood level.

No further action is required for sites within the bounds of the site parameter for flood level.

2.4.2.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification of floods and flood design considerations, and the associated acceptance criteria, are described in Section 2.4.2 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying floods are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 52.79(a)(1)(iii), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The related acceptance criteria are as follows:

- RG 1.59, "Design Basis Floods for Nuclear Power Plants" (NRC 1977a) as supplemented by best current practices
- RG 1.102, "Flood Protection for Nuclear Power Plants" (NRC 1976b).

2.4.2.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.2 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the site-specific flooding description. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

2.4.2.4.1 Flood History

Information Submitted by the Applicant

The applicant stated that historical measurements of gauge heights and/or discharges are available at five USGS stations near the LNP site. These stations and their records, reported by the applicant in the FSAR, are summarized in Table 2.4.2-1.

Table 2.4.2-1. Historical Flood Measurements Near the LNP Site

Name (USGS ID)	Stage Measurement (Maximum stage on date)*	Discharge Measurement	Comment
Withlacoochee River at Dunnellon, Florida (02313200)	1963–2007 (9.26 m (30.37 ft) NDVD29 on 9/27/2004)		Discharge data not available
Withlacoochee River at Inglis Dam near Dunnellon, Florida (02313230)	1985–2007 (8.54 m (28.03 ft) NGVD29 on 3/27/2005)	1969–2007	
Withlacoochee River below Inglis Dam near Dunnellon, Florida (02313231)	1969–2007 (2.82 m (9.25 ft) NGVD29 on 3/20/1998)		Discharge data not available
Withlacoochee River Bypass Channel near Dunnellon, Florida (02313250)	1971–2007 (8.57 m (28.11 ft) NGVD29 on 1/2/1994)	1970–2007	
Withlacoochee River at Chambers near Yankeetown, Florida (02313272)	2005–2007 (1.36 m (4.47 ft) NAVD88 during high tides on 6/13/2006 and 0.14 m (0.46 ft) NAVD88 during low tides on 3/21/2006)		Discharge data not available

* As noted previously, the staff independently verified that elevations near the LNP site referring to the NGVD29 datum are 0.31 m (1 ft) greater than those referring to the NAVD88 datum.

The applicant stated that the National Weather Service (NWS) Advanced Hydrologic Prediction Service (AHPS) has identified a flood stage of 8.8 m (29 ft), a moderate flood stage of 9.1 m (30 ft) NGVD29, and a major flood stage of 9.4 m (31 ft) all with respect to gauge datum for the USGS station 02313200, Withlacoochee River at Dunnellon, Florida. The applicant stated that during 1963–2007, the major flood stage has not been exceeded at this gauge, the moderate flood stage was exceeded for 22 days during September 27 – October 18, 2004, and the flood stage has been exceeded for 15 of the 44 years of record. Based on historical data, the applicant concluded that flooding at the LNP site is unlikely but lower elevation areas near Lake Rousseau, the Withlacoochee River, and the CFBC may become flooded during periods of high water.

NRC Staff's Technical Evaluation

The information presented in this section describes the NRC staff's review of information and analyses by the applicant and presented in LNP FSAR Section 2.4.2. The NRC staff's independent analysis, where needed for the review, is also included. An accurate description of historical flooding, flooding mechanisms, and combination of these mechanisms and a thorough analysis of the effects of local intense precipitation on the proposed site is needed for the staff to complete its safety review. To understand the process used to determine the design basis flood, the staff issued **RAI 02.04.02-01**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, the applicant should include information concerning design basis flooding at the plant site, including consideration of appropriate combinations of individual flooding mechanisms in addition to the most severe effects from individual mechanisms themselves. Please describe the process followed to determine the conceptual models for floods from local intense precipitation, probable maximum flood in the drainage area upstream of the site, surges, seiche, tsunami, seismically induced dam failures, landslides, and ice effects to ensure that the design basis flood is based on the most conservative of plausible conceptual models.

The applicant responded to the staff's RAI 02.04.02-01 in a letter dated July 13, 2009 (ML091950612). The applicant stated that conceptual site models were developed to estimate flooding from local intense precipitation, flooding in streams and rivers, flooding from upstream dam failures, flooding from surges and seiches, flooding from tsunami, flooding from landslides, and flooding from ice effects. The applicant used a runoff coefficient of 1.0, or an equivalent assumption of no precipitation loss to maximize the runoff from the local intense precipitation on the plant area. The applicant assumed that all stormwater conveyance features, including ditches, sewers, and culverts, would be non-functional during the local intense precipitation event. The applicant conceptualized that runoff from the plant area during the local intense precipitation event would be delivered offsite as flow over broad-crested weirs at downstream control points such as peripheral roads. Using this conceptualization, the applicant estimated the backwater profile to determine the maximum water surface elevations at the SSCs important to safety. The applicant described the conceptual models for other flooding mechanisms in the respective FSAR sections.

The staff reviewed the applicant's response to RAI 02.04.02-01 and determined that the applicant postulated a conservative conceptual model of flooding during local intense precipitation because it used no precipitation losses and used downstream controls to estimate backwater effects. The staff determined, therefore, that the applicant has provided sufficient information for the staff's independent review.

An accurate description of the history of flooding in the site area and adjacent region is required for the staff to perform its safety assessment. To analyze the history of flooding at the site, the staff used the information provided by the applicant and supplemented it with publicly available sources of information and field observations from the safety audit.

To review the historical floods near the LNP site, the staff independently obtained peak streamflow data from USGS real-time and historical stream gauges. The location of these gauges is shown in Figure 2.4.2-1.

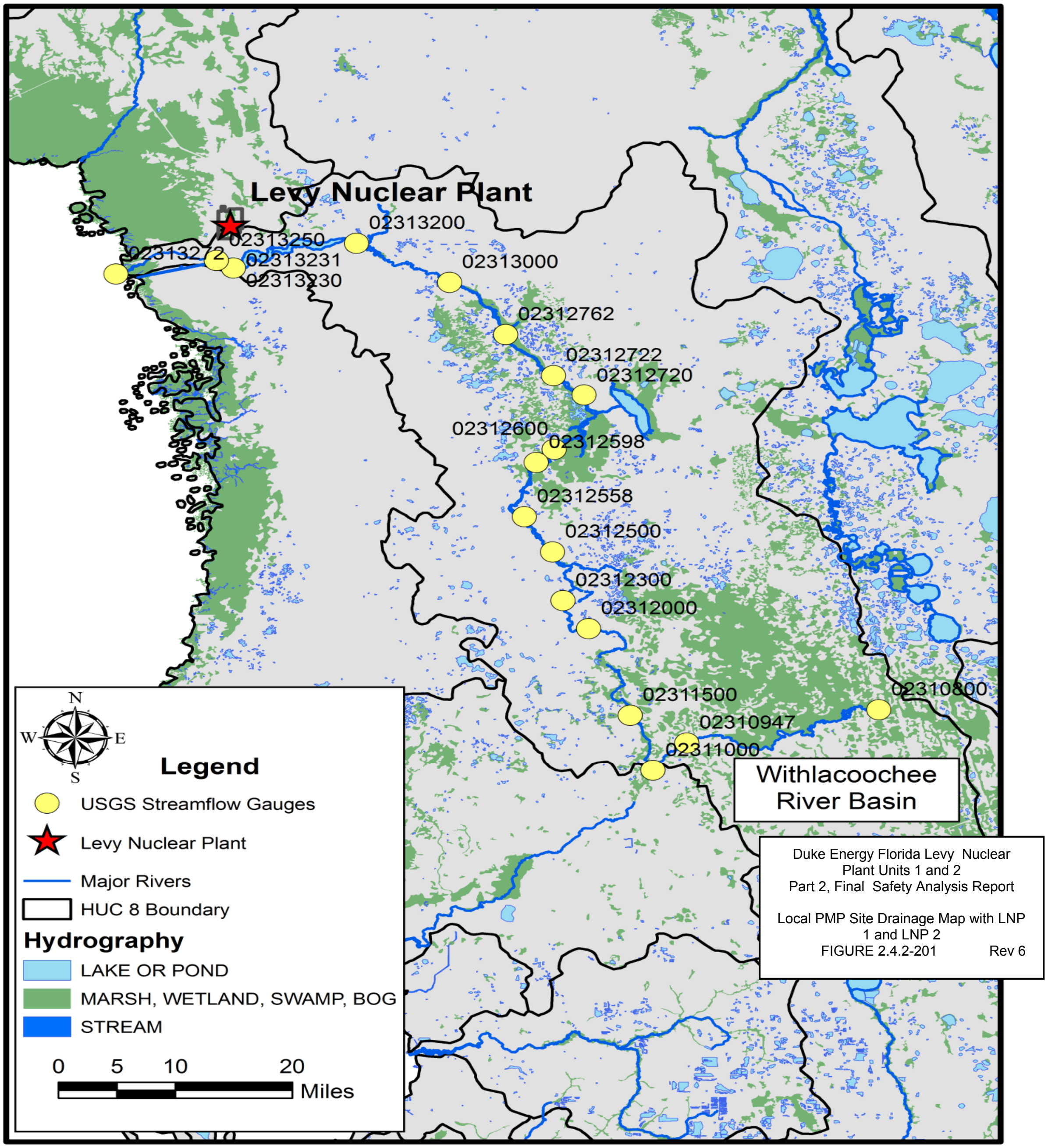


Figure 2.4.2-1. The Withlacoochee River Basin and USGS Streamflow Gauges

These gauges are located in the Withlacoochee River Basin. There are no gauges in the Spring Run and Thousandmile Creek-Halverson Creek Frontal subbasins of the Waccasassa River Basin. The staff reviewed the location of these gauges and determined that the gauges that represent flooding conditions most appropriately near the LNP site are: (1) USGS Gauge Number 02313200, Withlacoochee River at Dunnellon, Florida, (2) USGS Gauge Number 02313230, Withlacoochee River at Inglis Dam near Dunnellon, Florida, (3) USGS Gauge Number 02313231, Withlacoochee River below Inglis Dam near Dunnellon, Florida, and (4) USGS Gauge Number 02313250, Withlacoochee River Bypass Channel near Inglis, Florida. The staff summarized the records and data available at these USGS gauges and is presented in Table 2.4.2-2.

Table 2.4.2-2. Staff-Obtained Historical Flood Records for USGS Streamflow Gauges near the LNP Site

Name (USGS ID)	Stage Measurement (Maximum stage on date)	Peak Discharge Measurement (Maximum discharge on date)	Comment
Withlacoochee River at Dunnellon, Florida (02313200)	Since February 6, 1963 (9.26 m [30.37 ft] NGVD29 on September 27, 2004)		Data available for gauge height only
Withlacoochee River at Inglis Dam near Dunnellon, Florida (02313230)	Since October 1, 1985 (8.62 m [28.28 ft] NGVD29 on June 19, 1982)	Since 1970 Water-Year (171 m^3/s [6,030 cfs] on October 19, 2004)	Maximum stage and maximum discharge occurred on different dates
Withlacoochee River below Inglis Dam near Dunnellon, Florida (02313231)	Since October 1, 1969 (2.82 m [9.25 ft] NGVD29 on March 20, 1998)		Data available for gauge height only
Withlacoochee River Bypass Channel near Dunnellon, Florida (02313250)	Since September 9, 1971 (8.63 m [28.31 ft] NGVD29 on May 19, 1977)	Since 1970 Water-Year (52 m^3/s [1,840 cfs] on October 1, 1987)	Maximum stage and maximum discharge occurred on different dates

The staff concluded, based on available historical flood data at USGS streamflow gauges, that the finished grade elevation of the LNP site would be located approximately 6.1 m (20 ft) above the highest observed floodwater surface elevation in the Withlacoochee River near the site.

The staff also obtained historical gauge height data from NWS AHPS for Withlacoochee River at Dunnellon and Holder. The NWS AHPS website (2011) reported that the historical crests of the Withlacoochee River at Dunnellon show three instances when the flood stage exceeded the major flood stage of 9.4 m (31 ft) above gauge datum: 10.1 m (33 ft) on April 1, 1960, 9.6 m (31.6 ft) on October 12, 1961, and 9.59 m (31.45 ft) on July 17, 1934. The staff found that the NWS AHPS reported Withlacoochee River at Holder exceeding major flood stage of 3.35 m (11 ft) above gauge datum on five occasions: 4.05 m (13.28 ft) on April 5, 1960, 3.67 m (12.05 ft) on October 10, 1960, 3.54 m (11.63 ft) on July 8, 1934, 3.43 m (11.25 ft) on October 13, 2004, and 3.40 m (11.17 ft) on September 26, 1933. The NWS AHPS website does not report data for the other USGS gauges shown in Table 2.4.2-2. Because the Withlacoochee River at Dunnellon is the nearer location where NWS AHPS data is available, the staff used this location in its independent assessment. Based on the data reported by the NWS AHPS, the staff determined that the Withlacoochee River does occasionally exceed major flood stage. However, the highest reported stage for the river at Dunnellon is approximately 5.5 m (18 ft) below the proposed grade elevation of the LNP site. Based on its independent assessment, the staff determined that the LNP site has not been flooded by the Withlacoochee River during the period stream discharge and stage data have been recorded.

2.4.2.4.2 Flood Design Considerations

Information Submitted by the Applicant

The applicant stated that safety-related SSCs at the LNP site are protected against floods and flood waves caused by probable maximum events. Seismic Category I SSCs within the plant are designed for flooding due to natural phenomena and the basemat and exterior walls of these structures are designed for upward and lateral pressures from probable maximum flood (PMF) and high groundwater levels. The applicant has also stated that because the plant will be sited at a higher finished grade, no dynamic water forces will occur and that the finished grade will be adequately sloped to prevent dynamic forces associated with the probable maximum precipitation (PMP).

The applicant estimated the design basis flood elevation at the LNP site to be 15.17 m (49.78 ft) NAVD88 and it results from a probable maximum storm surge combined with wind-induced setup.

NRC Staff's Technical Evaluation

An accurate description of flooding mechanisms and combinations of these is required for the NRC staff to perform its safety assessment.

The NRC staff reviewed the applicant's responses to the RAIs to determine whether the process followed by the applicant to determine the design-basis flood is adequate. The NRC staff also used observations from its safety audit site tour and other independent data sources in its safety review. To analyze the effects of hydrodynamic forces on SSCs, the staff issued **RAI 02.04.02-02**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17 and 10 CFR Part 100, the applicant should include a determination of the capacity of site drainage facilities. Section 2.4.2.2 of the FSAR states "No dynamic water forces associated with high water levels will occur because of a higher finished plant grade. The dynamic forces associated with the probable maximum precipitation (PMP) are not factors in the analysis or design because the finished grade will be

> adequately sloped." Please clarify how sloping of the grade excludes consideration of dynamic forces in the analysis and design of safety-related SSCs during the local PMF event or provide an analysis that shows safety-related SSCs would be safe under the static and dynamic effects of the local PMF.

The applicant responded to the staff's RAI 02.04.02-02 in a letter dated July 13, 2009 (ML091950612). The applicant stated that the site grading would be performed such that the floor elevations of SSCs would be above the highest grade elevation. The applicant stated that the plant grade would be sloped away from the SSCs such that runoff would flow away from them. The applicant performed an analysis to estimate the water surface elevation during the local intense precipitation event and reported that the maximum water surface elevation including backwater effects would be less than the nominal plant grade floor elevation of 15.5 m (51 ft) NAVD88.

The staff reviewed the applicant's response and calculations performed to account for the backwater effects during the local intense precipitation event. As stated above, the applicant used a runoff coefficient of 1.0 for estimating the runoff from the local intense precipitation event. A runoff coefficient of 1.0 indicates that no infiltration or evapotranspiration losses were allowed and therefore, all of the precipitation contributed to runoff generation. This assumption resulted in maximization of runoff during the local intense precipitation event.

To perform the flooding analysis, the applicant divided the main plant area into seven drainage zones. The applicant estimated the time of concentration conservatively for each zone using Kirpich's Formula (Chow 1964). The applicant used the time of concentration to estimate the rainfall intensity, which is a parameter in the Rational Formula for peak discharge. The applicant represented the flow dynamics within the zones using a set of cross sections in the USACE HEC-RAS software. HEC-RAS was set up to simulate a steady-state backwater profile with the flow depth at the downstream boundary estimated using the broad-crested weir equation with the discharge set to the peak discharge estimated from the Rational Formula for the zone. The discharges at each of the cross sections were estimated by prorating the peak discharge for the zone by the ratio of contributing area upstream of the respective cross section to the total surface area of the zone. The staff determined that the applicant's approach is appropriate for estimation of water surface elevations near the safety-related SSCs because it considers the effects of the backwater flow profile upstream of the broad-crested weir that acts to control the depth of flow. Flow depths estimated from a steady-state hydraulic routing calculations envelop those from an unsteady hydraulic routing calculation if the peak discharges used in both simulations are the same. Therefore, the staff determined that the steady-state backwater profile would result in a conservative estimate of the greatest flow depth on the plant area during a transient local intense precipitation event.

The applicant used Manning's roughness coefficient values of 0.035 for peripheral areas and 0.025 for powerblock areas. The staff reviewed the Manning's roughness coefficients used by the applicant to determine whether they are appropriately conservative. The surface of the powerblock area would consist of concrete, asphalt pavement, or compacted gravel and grass. Chow (1959) recommends Manning's roughness coefficient ranges of 0.023 to 0.036 for gravel surfaces with dry rubble sides, a range of 0.013 to 0.016 for asphalt surface, and a range of 0.016 to 0.025 for straight and uniform earthen areas. The staff concluded that the applicant has used Manning's roughness coefficient values that correspond to the higher end of the recommended ranges. Higher Manning's roughness coefficient values result in higher water surface elevations. Therefore, the staff concluded that the applicant has conservatively estimated the floodwater surface elevation near the safety-related SSCs during the local intense precipitation event.

2.4.2.4.3 Effects of Local Intense Precipitation

Information Submitted by the Applicant

The applicant has also stated that water would not pond on safety-related SSCs of the LNP Units 1 and 2 because the roofs do not have drains or parapets and are sloped so rainfall is directed to gutters located along the edge of the roofs. The site drainage system is designed to drain runoff from a 50-year precipitation event to catch basins, underground pipes, or to open ditches. The drainage system is assumed to be non-functional during a local PMP event and the runoff from this event would be drained by overland flow on the ground surface away from safety-related SSCs to onsite retention ponds and eventually to the Lower Withlacoochee River and to the Gulf of Mexico.

Grading and drainage for the LNP site is shown in Figure 2.4.2-2. The LNP site is subdivided into seven drainage zones, A through G.

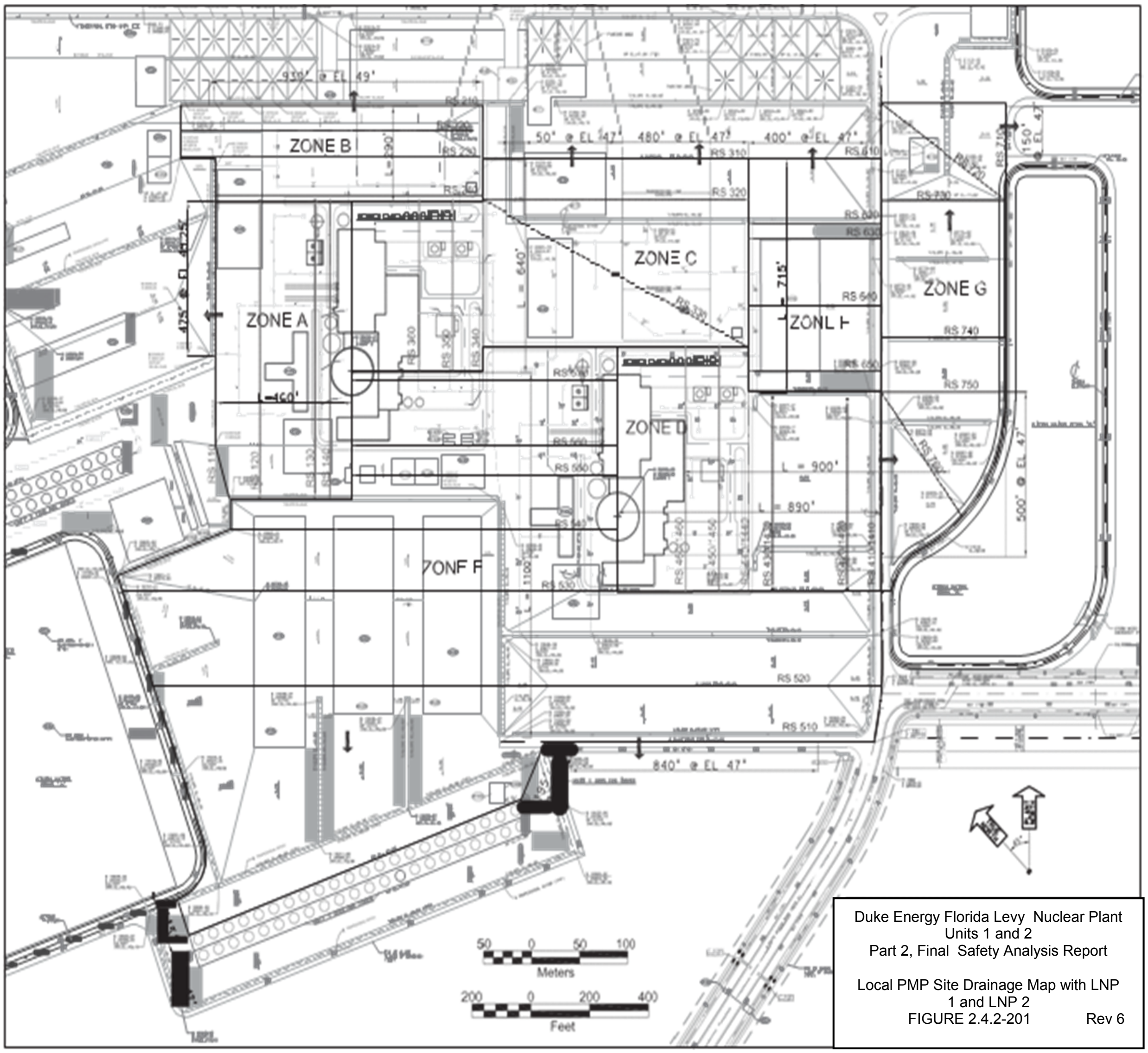


Figure 2.4.2-2. Local PMP Site Drainage Map with LNP 1 and LNP 2

The applicant determined the local PMP values for the LNP site using the procedure described in Hydrometeorological Report (HMR) No. 52 (Hansen et al. 1982). Local PMP values were taken as the 2.6-km^2 (1-mi^2) PMP values for durations ranging from 5 minutes to 24 hours. Table 2.4.2-3 shows the local PMP values estimated by the applicant.

Table 2.4.2-3. The Applicant-Estimated Probable Maximum Precipitation for the 2.6-km^2 (1-mi^2) Area

Duration		
Minutes	Hours	Precipitation (cm [in.])
5	0.08	15.95 (6.28)
15	0.25	24.92 (9.81)
30	0.5	36.37 (14.32)
60	1	49.80 (19.61)
360	6	94.51 (37.21)
720	12	114.91 (45.24)
1440	24	133.15 (52.42)

Runoff during the local PMP event was estimated using the rational method with the runoff coefficient set to 1.0. There are no safety-related facilities in drainage Zone G. The water levels for each of the other six drainage zones were estimated assuming that the peak runoff discharging out of the zone would behave as a discharge over a broad-crested weir. The water surface elevations estimated by the applicant for each of the other six zones are listed in Table 2.4.2-4.

Table 2.4.2-4. Maximum Water Surface Elevations on the LNP Site Estimated by the Applicant

Drainage Zone	Maximum Water Surface Elevation (m [ft] NAVD88)	Maximum Flow Velocity (m/s [ft/s])
A	15.3 (50.3)	0.4 (1.3)
B	15.3 (50.1)	0.6 (2.1)
C	15.5 (50.7)	1.1 (3.7)
D	15.4 (50.5)	0.6 (1.9)
E	15.4 (50.4)	0.8 (2.7)
F	15.4 (50.5)	1.2 (3.8)
D+G	15.4 (50.5)	1.0 (3.2)

In the FSAR, the applicant stated that roads in Zones A through F that may fall in the path of the overland flow during the local PMP event would be lowered to preclude safety-related facilities from being affected.

Based on the historical rainfall measured at the Ocala, Florida NWS Cooperative Station No. 086414, the applicant reported an annual mean precipitation of 126.19 cm (49.68 in.), a monthly mean precipitation range of 6.27 to 18.29 cm (2.47 to 7.20 in.), a highest monthly precipitation of 41.58 cm (16.37 in.) all recorded in April 1982, and a maximum daily precipitation of 29.77 cm (11.72 in.) recorded on April 8, 1982. The applicant stated that the LNP site is not expected to support long-term accumulation of ice and snow, and therefore, did not consider these as potential flooding mechanisms.

NRC Staff's Technical Evaluation

An accurate description of the method used to estimate local intense precipitation and the values obtained by the applicant is needed for the NRC staff to perform its safety assessment.

The NRC staff reviewed the applicant's responses to RAIs 2.4.2-1, 2.4.2-2, 2.4.2-3, and 2.4.2-4, which are discussed further in this section of the SER, to determine whether the effects of local intense precipitation considered by the applicant are adequate. The NRC staff also used observations from its safety audit site tour and other independent data sources in its safety review.

The staff independently estimated the local intense precipitation as the 1-hour, 2.6-square-km (1-square-mile) PMP from HMR 52 (Hansen et al. 1982). The staff-estimated local intense precipitation values are listed in Table 2.4.2-5.

Table 2.4.2-5. The Staff-Estimated Local Intense Precipitation at the LNP Site

Duration	Multiplier to 1-hour Precipitation Depth	Depth of Precipitation (cm [in.])
5 min	0.32 (HMR 52, Figure 36)	15.7 (6.2)
15 min	0.50 (HMR 52, Figure 37)	24.6 (9.7)
30 min	0.73 (HMR 52, Figure 38)	36.1 (14.2)
1 hour	1.0	49.3 (19.4)

The staff compared the applicant's estimate of the local intense precipitation with its own independent estimate. The applicant's estimates for the local intense precipitation are 1 percent higher than the staff's. The staff concluded that the applicant has appropriately and conservatively estimated the local intense precipitation at the LNP site. To obtain clarification regarding the site grade elevation and to determine the safety of SSCs, the staff issued **RAI 02.04.02-03**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17 and 10 CFR Part 100, the applicant should include a complete description of all spatial and temporal datasets used in support of its conclusions regarding safety of the plant. Data and descriptions should be sufficiently detailed to allow the staff to review the applicant's conclusions regarding the safety of the plant and to determine ~~of~~ the design bases of safety related SSCs. Please clarify if the stated site grade elevation of 15.5 m (51 ft) NGVD29 is subject to change.

The applicant responded to the staff's RAI 02.04.02-03 in a letter dated July 13, 2009 (ML091950612). The applicant stated that the nominal plant grade floor elevation of SSCs at the LNP site would be 15.5 m (51 ft) NAVD88 and is not subject to change. The staff used the nominal plant grade floor elevation of 15.5 m (51 ft) NAVD88 as the finished floor elevation of safety-related SSCs at the LNP site for all safety determinations in the hydrologic engineering sections of this report.

To determine the appropriateness of the methods used to estimate flood discharges and elevations during the local intense precipitation event, the staff issued **RAI 02.04.02-04**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, please clarify (1) the description of the methodology used to estimate the times of concentration for each drainage zone, (2) the locations and characteristics of the broad-crested weirs, and (3) the estimated backwater profile from the broad-crested weirs to the safety-related SSCs.

The applicant responded to the staff's RAI 02.04.02-04 in a letter dated July 13, 2009 (ML091950612). The applicant stated that the Kirpich Formula was used to estimate the time of concentration for each drainage zone. The Kirpich Formula uses the length of the drainage area measured along the flow and the average slope of the drainage area and is frequently used in design of urban drainage systems (Chow 1964). The staff concluded therefore, that the applicant's approach is appropriate.

The applicant described the location and characteristics of the broad-crested weirs used in the estimation of the floodwater surface elevation during the local intense precipitation event. The applicant stated that the broad-crested weirs are typically located at roads, tops of embankments, crests of site grades, or where the slope of the grade changes significantly. The applicant used the broad-crested weir equation (USACE 1987) to estimate the discharge over the weirs. The broad-crested weir equation uses a coefficient of discharge (USACE 1987). The staff reviewed the method described by USACE (1987) and the applicant's calculation package and determined that the applicant appropriately selected the discharge coefficient for the LNP site where the ratio of water depth over the broad-crested weir to the weir breadth is expected to be smaller than 0.5.

The applicant described its procedure for estimation of the backwater profiles for each of the seven runoff zones. Table 2.4.2-6 lists the characteristics of the runoff zones and the estimated flood properties during the local intense precipitation event.

Table 2.4.2-6. Characteristics of the Runoff Zones and Estimated Flood Properties

Runoff Zone	Area (ha [ac])	Peak Discharge (m^3/s [cfs])	Maximum Floodwater Surface Elevation (m [ft] NAVD88)	Maximum Flow Velocity (m/s [ft/s])
A	3.8 (9.4)	13.2 (465)	15.3 (50.3)	0.4 (1.3)
B	2.6 (6.5)	14.1 (499)	15.3 (50.1)	0.6 (2.1)
C	6.9 (17.0)	27.1 (957)	15.5 (50.7)	1.1 (3.7)
D	5.6 (13.9)	14.9 (525)	15.4 (50.5)	0.6 (1.9)
E	22.0 (54.3)	60.0 (2,120)	15.4 (50.4)	0.8 (2.7)
F	3.0 (7.3)	10.2 (361)	15.4 (50.5)	1.2 (3.8)
D+G	10.7 (26.4)	32.3 (1140)	15.4 (50.5)	1.0 (3.2)

Based on the review of the applicant's responses to the staff's RAIs, review of the applicant's calculation packages, and the staff's independent estimation of the local intense precipitation at the LNP site, the staff concluded that the applicant has adequately and conservatively estimated the effects of the local intense precipitation at the LNP site because (1) the local intense precipitation was conservatively estimated, (2) no precipitation losses were allowed, (3) an appropriate simulation model (HEC-RAS) was used, and (4) values used for Manning's roughness coefficients were conservative. The staff agrees with the applicant that the floodwater surface elevations in the powerblock area near the safety-related SSCs would not exceed the nominal plant grade floor elevation of 15.5 m (51 ft) NAVD88.

2.4.2.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.2.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed the information related to individual types of flood-producing phenomena, and combinations of flood-producing phenomena, considered in establishing the flood design bases for safety-related plant features. The information also covered the potential effects of local intense precipitation. The staff also confirmed that there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.2 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses COL Information Item 2.4-2.

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

2.4.3.1 *Introduction*

FSAR Section 2.4.3 describes the hydrological site characteristics affecting any potential hazard to the plant's safety-related facilities as a result of the effect of the PMF on streams and rivers. Section 2.4.3 of this SER provides a review of the following specific areas: (1) design basis for flooding in streams and rivers, (2) design basis for site drainage, (3) consideration of other site-related evaluation criteria, and (4) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.3.2 *Summary of Application*

This section of the COL FSAR addresses the site-specific information about PMFs on streams and rivers. The applicant addressed the information as follows:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.4-2

This section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 19 of the AP1000 DCD.

> The COL applicants referencing the AP1000 design will address the following site-specific information on historical flooding and potential flooding factors, including the effects of local intense precipitation:
>
> - Probable Maximum Flood on Streams and Rivers – Site-specific information that will be used to determine design-basis flooding at the site. This information will include the PMF on streams and rivers.
> - Dam Failures – Site-specific information on potential dam failures.
> - Probable Maximum Surge and Seiche Flooding – Site-specific information on probable maximum surge and seiche flooding.
> - Probable Maximum Tsunami Loading – Site-specific information on probable maximum tsunami loading.
> - Flood Protection Requirements – Site-specific information on flood protection requirements or verification that flood protection is not required to meet the site parameter for flood level.
>
> No further action is required for sites within the bounds of the site parameter for flood level.

2.4.3.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification of floods and flood design considerations, and the associated acceptance criteria, are described in Section 2.4.3 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying probable maximum flooding on streams and rivers are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The related acceptance criteria are as follows:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants" (NRC 1976a).
- RG 1.29, "Seismic Design Classification" (NRC 2007b).
- RG 1.59, "Design Basis Floods for Nuclear Power Plants" (NRC 1977a) as supplemented by best current practices.
- RG 1.102, "Flood Protection for Nuclear Power Plants" (NRC 1976b).
- RG 1.206 "Combined License Applications for Nuclear Power Plants (LWR Edition)" (NRC 2007c).

2.4.3.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.3 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the site-specific PMF on streams and rivers. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

An accurate description of the assessment of the PMF level is needed for the staff to perform its safety assessment. To understand the process followed in the analysis of in-stream flooding, the staff issued **RAI 02.04.03-01**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the following characteristics are needed, and should be based on conservative assumptions of hydrometeorologic characteristics in the drainage area: (a) the area of the watershed used to estimate flooding in streams and rivers, (b) the total depth of PMP and the PMP hyetograph, (c) the maximum PMF water surface elevation in streams and rivers with coincident wind-waves, and (d) hydraulic characteristics that describe dynamic effects of PMF on SSCs important to safety. Please describe the process followed to determine the conceptual models for floods in streams and rivers and in site drainage system to ensure that the design basis flood is based on the most conservative of plausible conceptual models.

The applicant responded to the staff's RAI 02.04.03-01 in a letter dated June 23, 2009 (ML091760626). The applicant stated that the LNP safety-related SSCs would be located entirely in the Waccasassa River Basin and would also be located away from nearby waterbodies. The applicant also stated that because there are no named streams on the LNP site and because there are no known water-control structures in the Waccasassa River Basin, safety-related SSCs of the LNP units would not be affected by flooding in the Waccasassa River Basin. The runoff from the LNP site drains to the southwest towards the Lower Withlacoochee River and the Gulf of Mexico. The Withlacoochee River and Lake Rousseau are located approximately 4.8 km (3 mi) south of the LNP site and are located in the Withlacoochee River Basin, which is hydrologically separated from the Waccasassa River Basin.

The applicant stated that to determine the design basis flood, it used guidance provided by NRC RGs 1.206 and 1.59 and American National Standards Institute (ANSI)/American Nuclear Society (ANS)-2.8-1992. The applicant considered the Withlacoochee River Basin upstream of the Inglis Dam as the drainage area for determination of the PMF. The Withlacoochee River Basin above Inglis Dam was divided into 18 subbasins. The applicant estimated the PMP over the basin using the procedures described in HMRs 51 and 52 and ANSI/ANS-2.8-1992. The applicant used a PMP storm lasting 9 days; an antecedent storm, with 40 percent of the estimated PMP depths, was used during the first 3 days; the middle 3 days were dry (no precipitation); and the full PMP storm occurred during last 3 days.

The applicant described its approach for determining the PMF in the Withlacoochee River Basin to determine whether the LNP site may be affected by it. The drainage area of the Withlacoochee River Basin is approximately 5,232 km^2 (2,020 mi^2). The applicant estimated the PMP over the Withlacoochee River Basin for determination of the PMF. The PMF water surface elevation in Lake Rousseau was determined to be 9.1 m (29.7 ft) NAVD88 and the plant grade floor elevation of LNP SSCs would be at 15.5 m (51 ft) NAVD88. The applicant concluded that there is a substantial margin, 6.5 m (21.3 ft), between the plant grade floor elevation of LNP SSCs and the maximum PMF water surface elevation in Lake Rousseau.

The applicant used unit hydrographs to determine the runoff from the PMP storm for each subbasin of the Withlacoochee River Basin above Inglis Dam. The applicant used no initial loss. The applicant used a constant loss rate during the PMP storm. The runoff hydrograph from each subbasin was routed using the Muskingum routing method in the stream reaches to determine the inflow hydrograph to Lake Rousseau. The inflow to Lake Rousseau was routed through the lake using its stage-storage-discharge relationship and characteristics of the outlet works.

The staff reviewed the applicant's response to RAI 02.04.03-01 and determined that the applicant has provided sufficient information regarding the conceptual models used in the FSAR analyses. The staff agrees with the applicant that there are no streams or rivers of sufficient size in the Spring Run and Thousandmile Creek-Halverson Creek Frontal subbasins of the Waccasassa River Basin to pose a flooding hazard to SSCs at the LNP site. The overland flow in these Frontal subbasins resulting from the local intense precipitation would flow generally southwest. Because the existing grade elevation at the proposed location of the LNP units' powerblock area would be raised, the staff concluded that the floodwater surface elevation produced by the local intense precipitation at the LNP site, presented by the applicant in FSAR

Section 2.4.2 is appropriate. The staff also agrees with the applicant that the most conservative scenario for flooding in streams and rivers that may pose a hazard at the LNP site would occur from a PMF in the adjoining Withlacoochee River Basin. Therefore the staff concluded that the applicant has correctly and conservatively identified the alternative conceptual models for flooding in river and streams near the LNP site.

2.4.3.4.1 Probable Maximum Precipitation

Information Submitted by the Applicant

The applicant estimated the generalized cumulative PMP depths for different areas and durations from HMR 51 (Schreiner and Riedel 1978). The drainage area of the Withlacoochee River Basin upstream of the Inglis Dam was estimated to be 5,232 km^2 (2,020 mi^2). From the cumulative PMP depths for various area sizes, the applicant estimated the 6-hour incremental PMP depths.

The preferred orientation of the PMP isohyetal pattern from HMR 52 (Hansen et al. 1982) is 205º. The applicant estimated that the PMP isohyetal pattern that produced the maximum volume of precipitation within the Withlacoochee River Basin was 150º (Figure 2.4.3-1 [adapted from FSAR Rev 0 Figure 2.4.3-205]). Because the difference in orientation between the preferred and the maximum-volume orientation directions exceeds 40º, the applicant adjusted the incremental PMP depths, which resulted in a small decrease in the unadjusted incremental values.

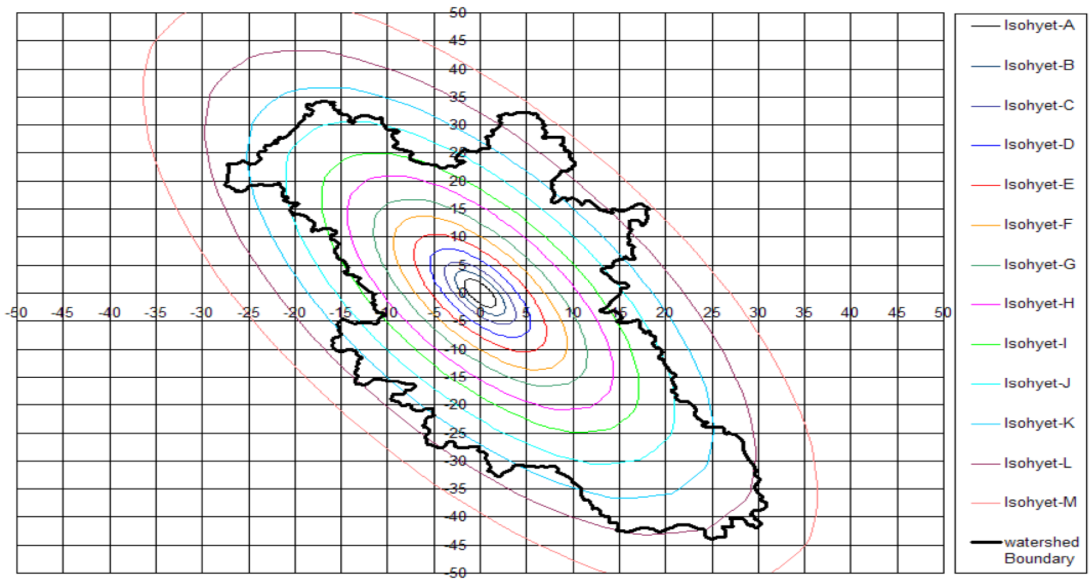
The applicant estimated the values of the isohyets corresponding to the maximum precipitation volume within the Withlacoochee River Basin for the three 6-hour durations with the highest incremental precipitation using the procedure described in HMR 52 (Hansen et al. 1982). The PMP spatial pattern size that maximized the precipitation in the basin was determined to be 3,885 km^2, (1,500 mi^2). Based on this PMP isohyetal pattern, the applicant estimated the basin-average incremental precipitation depths for each of the twelve 6-hour durations. Table 2.4.3-1 lists the 72-hour basin-average PMP for the Withlacoochee River Basin.

The applicant developed the 216-hour, or 9-day design storm for the Withlacoochee River Basin using a 72-hour antecedent storm at 40 percent of the PMP depths shown in Table 2.4.3-1, followed by a 72-hour period of no rain, and the last 72-hour period with precipitation values rearranged from those shown in the last column of Table 2.4.3-1 (100 percent PMP).

NRC Staff's Technical Evaluation

The staff reviewed the applicant's analysis for the estimation of PMP in the Withlacoochee River Basin above Inglis Dam. The staff independently estimated the PMP following the procedures described in HMRs 51 (Schreiner and Riedel 1978) and 52 (Hansen et al. 1982) to verify the applicant's PMP estimates. The staff-estimated PMP depths agree with the applicant's estimates. The staff concluded, therefore, that the applicant has correctly and conservatively estimated the PMP in the Withlacoochee River Basin above Inglis Dam.

LNP COL 2.4-2



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Orientation of the Storm Pattern for Maximum Precipitation Volume

FIGURE 2.4.3-205 Rev 0

Figure 2.4.3-1. Spatial Pattern of PMP Storm over the Withlacoochee River Basin

Table 2.4.3-1. The 72-hour Basin-Average PMP for the Withlacoochee River Basin Estimated by the Applicant

Six-hour Duration	Time Since Beginning of the PMP Storm (hr)	Cumulative PMP Depth (cm [in.])	Incremental PMP Depth (cm [in.])
1	6	36.12 (14.22)	36.12 (14.22)
2	12	52.86 (20.81)	16.74 (6.59)
3	18	62.61 (24.65)	9.75 (3.84)
4	24	69.22 (27.25)	6.60 (2.60)
5	30	74.09 (29.17)	4.88 (1.92)
6	36	77.93 (30.68)	3.81 (1.50)
7	42	81.00 (31.89)	3.10 (1.22)
8	48	83.59 (32.91)	2.59 (1.02)
9	54	85.80 (33.78)	2.21 (0.87)
10	60	87.70 (34.53)	1.91 (0.75)
11	66	89.36 (35.18)	1.68 (0.66)
12	72	90.86 (35.77)	1.47 (0.58)

2.4.3.4.2 Precipitation Losses

Information Submitted by the Applicant

The applicant estimated the initial and constant loss rates, which are used by the HEC-HMS computer model and are based on the recommendations of the Federal Energy Regulatory Commission (FERC). The applicant assumed that the entire Withlacoochee River Basin would have saturated soils at the start of the PMP storm, that there would be no initial loss, and that the constant loss during the PMP storm would occur at the minimum rate. The applicant used soils data for the Withlacoochee River Basin available from the SWFWMD to estimate the soil hydrologic groups for each of the subbasins. U.S. Department of Agriculture (USDA) NRCS recommendations (NRCS 1986) for minimum infiltration rates were used for each soil hydrologic group to estimate area-weighted average for each subbasin.

NRC Staff's Technical Evaluation

The staff reviewed the loss rates used by the applicant in its PMF estimation. The staff determined, using a review of the applicant's calculations, that no initial loss was applied to the

PMP storm. The assumption of no initial loss is conservative because it maximizes runoff. However, the applicant used a constant loss rate for the duration of the PMP storm under consideration. The constant loss rate varies, depending on soil type in different parts of the Withlacoochee River Basin. The loss rates ranged from 0.13 to 0.74 cm/h (0.05 to 0.29 in/h). During a PMP storm, especially when an antecedent storm, 40 percent of the PMP occurs prior to the full PMP storm, the soils in the basin would be close to saturation and therefore would only support minimal continuing loss rates. The staff reviewed the applicant's method of estimating the constant loss rate based on spatial distribution of soils in the subbasins. The staff agrees that the applicant's approach is reasonable and conservative because it accounts for subbasin-specific conditions and uses minimum infiltration rates for the different hydrologic soil groups, respectively.

2.4.3.4.3 Runoff and Stream Course Models

Information Submitted by the Applicant

The applicant subdivided the Withlacoochee River Basin into 18 subbasins. Lake Rousseau was assumed to be the 19th subbasin.

Runoff from the subbasins was estimated using a unit hydrograph approach based on Snyder's synthetic unit graphs. Some of the parameters for the Snyder's unit hydrograph were obtained from subbasin geometry; these include the flow path length from outlet to the hydraulically farthest point L and the length of flow path from outlet to centroid of the subbasin L_c. Other parameters were obtained from literature and these include the lag coefficient C_t and the peaking coefficient C_p.

The mean monthly discharge in the Withlacoochee River at USGS gauge 02313000 was used as the baseflow. Muskingum routing was used for streams. The applicant used a trial-and-error procedure to estimate the parameters of the Muskingum routing method. First, the applicant obtained an estimate of 10-, 25-, 50-, and 100-year return period flood discharges at USGS gauge 02313000 using a Log-Pearson Type III distribution subsequently adjusted for the difference in drainage areas at USGS gauge 02313000 and that for the whole Withlacoochee River Basin. The applicant estimated a precipitation-discharge relationship using 24-hour rainfall data for the same return periods. The applicant used the precipitation-discharge relationship to estimate the 500-year and the standard project rainfall amounts. The applicant applied the HEC-HMS model to reproduce the 10-year, 25-year, 50-year, 100-year, 500-year, and the standard project floods using previously estimated rainfall rates and by varying the Muskingum routing parameters.

The applicant used Lake Rousseau bathymetry data from a commercial source and the USGS digital terrain data to develop stage-storage curve for the lake. The applicant obtained the stage-discharge relationships for the Inglis Dam and the Inglis Lock from the State of Florida Environmental Protection Agency. The low-lying area around Inglis Dam was considered to act as an ogee spillway.

NRC Staff's Technical Evaluation

The staff reviewed the methodology adopted by the applicant in the development of the stream course model. The Withlacoochee River Basin is generally flat and has a few storage areas within the basin. The applicant ignored the storage and detention capacity of these storage areas in the hydrologic model used to estimate the PMF. Ignoring the storage and detention capacity would lead to higher peak discharges and quicker runoff response within the basin because precipitation excess would not be retained or detained by these storage areas. The staff determined that the applicant has adequately presented delineations of the subbasins and the stream network within the Withlacoochee River Basin above the Inglis Dam. To obtain a clear understanding of the applicant's process to determine the design-basis flood using combinations of events, the staff issued **RAI 02.04.03-02**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, the applicant should include information concerning design basis flooding at the plant site, including consideration of appropriate combinations of individual flooding mechanisms in addition to the most severe effects from individual mechanisms themselves. Please clarify the combined events criterion used to identify the design basis flood at the LNP site and to explicitly state the value of the design basis flood in the FSAR including a description of any adjustment made for long-term sea level rise.

The applicant responded to staff's RAI 02.04.03-02 in a letter dated June 23, 2009 (ML091760626). The applicant stated that various flood scenarios involving Lake Rousseau, the Withlacoochee River, the CFBC, and the Gulf of Mexico were considered. The applicant stated that various individual flooding mechanisms as well as combinations of these, as described in ANSI/ANS-2.8-1992 were considered. The individual flooding events considered included precipitation- and snowmelt-induced floods, failures of dams and other water-control structures, landslides, storm surges, seiches, wind-wave action, ice jams, channel changes and blockages, tsunami, volcanic eruptions, and glaciers. Of these scenarios, the applicant stated that flooding from snowmelt, landslides, ice jams, volcanic eruptions, and glaciers were not considered because these events are unlikely at and near the LNP site.

The applicant stated that the combined events considered for estimation of design basis flood consisted of wind influence, seasonal compatibility, storm optimization, and reservoirs. The applicant stated that wind influence was not explicitly considered during the PMF analysis because the LNP site is located approximately 3 mi from Lake Rousseau. The applicant also did not consider seasonality in the PMF analysis but used an estimate of worst-case flood conditions. The applicant stated that the Withlacoochee River meanders through a broad, flat plain and the river basin contains several swamplands, marshes, ponds, and shallow lakes. The applicant stated that it did not consider any reservoirs or waterbodies upstream of Lake Rousseau because floodwaters in the basin would spread into marshlands and lowlands adjacent to the river channel.

The applicant stated that the design basis flood elevation for the LNP safety-related SSCs results from the storm surge caused by a probable maximum hurricane (PMH) in combination with 10 percent exceedance tides and wind-effects.

The applicant stated that it estimated the long-term sea level rise near the LNP site using data from the tidal gauge located at Cedar Key, Florida. The applicant stated that the upper 95 percent confidence bound of sea level rise at the Cedar Key, Florida, is 1.99 mm/yr (0.08 in/yr), which would result in a 60-year rise of approximately 0.1 m (0.4 ft).

The staff reviewed the applicant's response to RAI 02.04.03-02 and concluded that the applicant has provided sufficient information regarding the design basis floodwater surface elevation at the LNP site. However, in order to determine whether the applicant followed a clear, consistent, and conservative approach in characterizing the hydrometeorological and hydrological parameters, the staff issued **RAI 02.04.03-03**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the following characteristics are needed, and should be based on conservative assumptions of hydrometeorologic characteristics in the drainage area: (a) the area of the watershed used to estimate flooding in streams and rivers, (b) the total depth of PMP and the PMP hyetograph, (c) the maximum PMF water surface elevation in streams and rivers with coincident wind-waves, and (d) hydraulic characteristics that describe dynamic effects of PMF on SSCs important to safety. Please justify (1) the use of unit hydrograph method for estimating the runoff from precipitation falling on the surface of Lake Rousseau and (2) the appropriateness of Snyder's unit hydrograph under PMP conditions given the assumption of linearity in the unit hydrograph approach of runoff generation.

The applicant responded to the staff's RAI 02.04.03-03 in a letter dated June 23, 2009 (ML091760626). The applicant provided a justification for the use of a unit hydrograph for estimation of runoff from the surface of Lake Rousseau during the PMP event. The applicant presented the assumption behind the unit hydrograph theory. The applicant stated that the use of unit hydrograph theory is best suited for estimation of runoff from the surface of a lake because the assumption of the theory would be minimal. The applicant also suggested that because several unit hydrograph methods, such as the Single-Linear Reservoir method and the Nash method were conceptualized using a reservoir, the unit hydrograph theory should be applicable for runoff estimation from their surfaces.

The staff disagrees with this approach. The unit hydrograph (UH) theory is used to describe the time distribution of surface runoff at the outlet produced by a constant and uniform rainfall excess event over a watershed. The time delay and attenuation in discharge compared to the rainfall excess event occurs because of the physical obstruction to overland flow over the surface of the watershed. Within the watershed, overland flow also accumulates into channels and streams. Both of these characteristics (overland flow and presence of channels and streams) are not present when considering runoff from the surface of a lake or reservoir and therefore a UH is not an appropriate tool to describe its response to a rainfall event.

The applicant provided a set of justifications to support using unit hydrographs for drainage basins of large areas. The applicant stated that several storage areas exist within the Withlacoochee River Basin such as intermittent streams, connected lakes and wetlands, and sinkholes. The applicant stated that in drainage basins with large floodplains with vegetation and other obstructions within the overbank areas, average velocities are likely to remain fairly constant or even decrease to some extent as flow rate increases. The applicant concluded that this behavior would reduce nonlinearity effects.

The staff reviewed the applicant's response to RAI 2.4.3-3 and concluded that the applicant has provided no other supporting evidence, such as data from observed rainfall and runoff events that support this hypothesis. Generally, as discharge increases, flow depth increases, and therefore velocity of flow increases. The staff concluded that the applicant has not presented sufficient information to support the case that nonlinear response in the Withlacoochee River Basin is insignificant.

The applicant acknowledged that published literature recommends derivation of unit hydrographs from large historical storms if the intent is to apply the unit hydrograph for estimation of hypothetical floods such as the PMF from hypothetical storms, such as the PMP.

The applicant also quoted text from Sivapalan et al. (2002) to justify linear runoff response in the Withlacoochee River Basin. The same reference (Sivapalan et al. 2002) also includes this observation, that the applicant did not include in its response: "On the other hand, Robinson et al. [1995], using numerical simulations, showed that nonlinearity at small scales is dominated by the hillslope response, that nonlinearity at large scales is dominated by channel network hydrodynamics, and that nonlinearity does not really disappear at any scale."

The staff disagrees with the applicant that the response of the Withlacoochee River Basin can be considered linear. Because the applicant was not able to provide a technically sound and conservative assessment of the PMF in the Withlacoochee River Basin, the staff issued **RAI 02.04.03-05**, which states:

> In reply to the staff's RAI 2.4.3-03, the applicant stated that application of a UH to predict runoff from the surface of a reservoir is acceptable. The staff disagrees with this approach. The UH theory is used to describe the time distribution of surface runoff at the outlet produced by a constant and uniform rainfall excess event over a watershed. The time delay and attenuation in discharge compared to the rainfall excess event occurs because of the physical obstruction to overland flow over the surface of the watershed. Within the watershed, overland flow also accumulates into channels and streams. Both of these characteristics (overland flow and presence of channels and streams) are not present when considering runoff from the surface of a lake or reservoir and therefore a UH is not an appropriate tool to describe its response to a rainfall event. The applicant should use a rainfall-runoff response function that is appropriate for the surface of Lake Rousseau.
>
> In reply to the staff's RAI 2.4.3-03, the applicant's response includes text quoted from Sivapalan et al. (2002). The same reference (Sivapalan et al. 2002) also includes this observation, that the applicant did not include in its response: "On

> the other hand, Robinson et al. [1995], using numerical simulations, showed that nonlinearity at small scales is dominated by the hillslope response, that nonlinearity at large scales is dominated by channel network hydrodynamics, and that nonlinearity does not really disappear at any scale." The staff disagrees with the applicant that the response of the Withlacoochee River Basin can be considered linear. The applicant should use UHs that are appropriately representative of overland flow and runoff generation conditions in the basin and conservative in predicting the discharge in the Withlacoochee River at the time a PMP event is likely to occur.

The applicant responded to the staff's RAI 02.04.03-05 in a letter dated June 18, 2010 (ML101740490). The applicant's reply to the staff's RAI presented justification for using a unit hydrograph for the surface area of Lake Rousseau. The applicant stated that using a unit hydrograph would result in a conservative estimate of the peak flood discharge because the lag times associated with upstream drainage areas is larger than a day. The staff agreed with the applicant that using a unit hydrograph for the surface area of Lake Rousseau would result in a more conservative discharge. The staff's review is required to ascertain that the analyses used to support safety conclusions in an FSAR are representative of the hydrologic characteristics of the study area in addition to being conservative and the staff believes that the applicant has not demonstrated this requirement conclusively for the study area. The staff also reviewed the applicant's sensitivity analysis used to determine whether the estimated unit hydrographs would accurately predict large flood events in the Withlacoochee River Basin. While the staff agreed with the applicant that its unit hydrographs estimate peak discharge of relatively large floods conservatively, the staff found that the applicant had not applied all literature recommendations for adjustment of unit hydrographs for application to extremely large floods approaching the PMF. To resolve the outstanding questions with regard to the PMF analysis and the appropriate choice of representative parameters, the staff issued **RAI 02.04.03-06**, which states:

> In RAI 2.4.3-05 (RAI ID 4628, Question 17566), the staff requested the applicant to provide a probable maximum flood (PMF) analysis for the Withlacoochee River watershed that used (1) an appropriate rainfall-runoff response function for Lake Rousseau and (2) unit hydrographs for the subbasins of the Withlacoochee River watershed that are appropriately representative of overland flow and runoff generation conditions in the basin and conservative in predicting the discharge in the Withlacoochee River at the time a probable maximum precipitation (PMP) event is likely to occur.
>
> The applicant's response, dated June 18, 2010, stated that the applicant's approach to a unit hydrograph for generation of runoff from the precipitation falling on the surface of Lake Rousseau would result in a conservative estimate of the probable maximum flood because the lag times associated with subbasins upstream of Lake Rousseau are larger than a day. Therefore, the applicant stated that use of the alternative approach of assuming no lag in generation of runoff from precipitation falling on the surface of Lake Rousseau would not be conservative because peak runoff from the upstream subbasins would not coincide with the peak runoff from Lake Rousseau. While NRC agrees that using a unit hydrograph for Lake Rousseau would be more conservative, the analysis

that supports safety conclusions in the FSAR must be representative of the hydrologic characteristics of the study area, in addition to being conservative. The applicant must provide an appropriate rainfall-runoff response function for Lake Rousseau and update the PMF analysis based on this response function.

The applicant's June 18, 2010, response also described a sensitivity analysis that was performed to determine the ability of the subbasin unit hydrographs to predict large floods including the standard project flood. The applicant stated that Snyder peak coefficient, the parameter C_p, was increased from its regional value of 0.6 to 0.8, a 33 percent increase that would result in a corresponding increase of 33 percent to peak discharge. The FSAR Rev 1 Table 2.4.3-221 shows that a C_p value of 0.8 was used for all subbasins. However, the text in FSAR Rev 1 Section 2.4.3.3.1 states that a value of 0.6 was used for C_p.

While the applicant has demonstrated that the unit hydrographs it employs estimate the peak discharge of relatively large floods conservatively, the literature guidance also recommends reduction in time to peak for the unit hydrographs that are used to predict large floods such as the PMF. NRC requests that the applicant:

(1) verify that the value of Snyder peaking coefficient, C_p, used in the PMF analysis is 0.8

(2) adjust time to peak discharge appropriately for each subbasin unit hydrograph

(3) update the PMF analysis

(4) provide input files for the PMF analysis, and

(5) provide related updates to FSAR Section 2.4.3, ensuring that the text is consistent with the analysis performed.

The applicant responded to the staff's RAI 02.04.03-06 in a letter dated November 16, 2010 (ML103300096). The applicant stated that it used a direct runoff function with zero travel time to estimate the contribution from Lake Rousseau's surface. The applicant also verified that a C_p value of 0.8 was used in the PMF analysis and that the C_p value of 0.6 was just the base case reported in the FSAR. The applicant stated that it modified the subbasin unit hydrographs, except that for the surface area of Lake Rousseau by further increasing the peak discharges predicted by unit hydrographs obtained from setting C_p to 0.8 by 25 percent. The applicant also reduced the lag time, or the time to peak discharge of the unit hydrographs, as recommended in literature. The applicant re-estimated the PMF in the Withlacoochee River Basin after making the above changes to the unit hydrographs. The applicant provided text changes to the FSAR that will be incorporated in a future revision. The staff is tracking this proposed FSAR text change as **Confirmatory Item 2.4.3-1**.

Resolution of Confirmatory Item 2.4.3-1

Confirmatory Item 2.4.3-1 is an applicant commitment to update Section 2.4.3 of its FSAR. The staff verified that LNP COL FSAR Section 2.4.3 was appropriately updated. As a result, Confirmatory Item 2.4.3-1 is now closed.

The staff reviewed the applicant's response to RAI 02.04.03-06 and determined that the applicant has chosen to use characterizations that are consistent with the hydrologic characteristics in the Withlacoochee River Basin above the Inglis Dam, specifically the use of a direct discharge function for the surface area of Lake Rousseau. The staff also determined that the applicant has conservatively applied guidance available in literature to adjust unit hydrographs for use in prediction of floods approaching the magnitude of a PMF, specifically increasing the value of Cp and reducing the lag time. The applicant's revised PMF discharges showed a larger and earlier peak. The staff concluded therefore, that the applicant has used appropriate and conservative methods in the estimation of the PMF in the Withlacoochee River Basin above the Inglis Dam.

2.4.3.4.4 Probable Maximum Flood Flow

Information Submitted by the Applicant

The applicant estimated the PMF in the Withlacoochee River Basin using the HEC-HMS computer program with input using the estimated PMP in the basin, the loss rates described in Section 2.4.3.4.2 of this SER, and the unit hydrographs for the 19 subbasins. The applicant assumed that Lake Rousseau was full at the start of the PMP event in the Withlacoochee River Basin. The estimated peak PMF inflow into Lake Rousseau was 1,720 m^3/s (60,755 cfs) and it occurred 4 weeks after the start of the PMP event.

NRC Staff's Technical Evaluation

The staff reviewed the information related to estimation of probable maximum flood flow that was provided by the applicant. To determine that the parameters used in the estimation of PMF flow are representative of the hydrometeorological conditions and demonstrate the required level of conservatism, the staff issued **RAI 02.04.03-04**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the following characteristics are needed, and should be based on conservative assumptions of hydrometeorologic characteristics in the drainage area: (a) the area of the watershed used to estimate flooding in streams and rivers, (b) the total depth of PMP and the PMP hyetograph, (c) the maximum PMF water surface elevation in streams and rivers with coincident wind-waves, and (d) hydraulic characteristics that describe dynamic effects of PMF on SSCs important to safety. Please clarify the estimation of base flow used in the determination of the PMF discharge.

The applicant responded to the staff's RAI 02.04.03-04 in a letter dated June 23, 2009 (ML091760626). The applicant stated that ANSI/ANS-2.8-1992 recommends that the mean monthly flow during the month of occurrence of the PMF should be used as the baseflow. The applicant stated that because seasonality was not considered in the PMP and subsequent PMF estimations, the mean annual flow was assumed to be the baseflow. The baseflow used was 28.5 m^3/s (1,008 cfs), which was estimated from monthly streamflow statistics published by the USGS for the streamflow gage 02313000, Withlacoochee River near Holder. The applicant also presented mean monthly flow values at this streamflow gauge. The mean monthly streamflow at the Holder gauge varies from 16.1 m^3/s (570 cfs) in June to 46.1 m^3/s (1627 cfs) in September. The applicant also performed an analysis by using mean monthly flow for the months of August through November (mean monthly flow for these months are 35.2, 46.1, 45.8, and 29.1 m^3/s (1,243, 1,627, 1,617, and 1,029 cfs), respectively) to investigate the sensitivity of the PMF water surface elevation. The PMF water surface elevation changed less than 0.03 m (a tenth of a foot). The applicant concluded that the PMF water surface elevation is insensitive to baseflow.

The staff reviewed the descriptions and analysis details provided by the applicant and determined that the applicant has provided sufficient information regarding baseflow in the Withlacoochee River.

2.4.3.4.5 Water Level Determinations

Information Submitted by the Applicant

The applicant estimated the water surface elevations in Lake Rousseau using the HEC-HMS computer program input with the estimated inflow into Lake Rousseau and the Lake Rousseau stage-storage and stage-discharge relationships. The applicant conservatively assumed that the spillway gates on the Inglis Dam would be inoperable during the PMF event. Under these conditions, the applicant estimated that the maximum water surface elevation in Lake Rousseau would be 9.1 m (29.7 ft) NAVD88.

NRC Staff's Technical Evaluation

The staff reviewed the methodology adopted by the applicant in estimation of water surface elevations in Lake Rousseau under the PMF scenario. The staff agrees that the applicant has applied appropriate methods by specifically using the HEC-HMS computer program to route the PMF discharge through Lake Rousseau. The staff also agrees that the applicant has used conservative conditions, specifically the assumption that spillway gates on the Inglis Dam would be inoperable during the PMF event. Therefore, the staff concluded that the applicant has conservatively estimated the maximum water surface elevation in Lake Rousseau during the PMF event. The applicant-estimated maximum water surface elevation in Lake Rousseau during the PMF event—9.1 m (29.7 ft) NAVD88—is significantly lower than the nominal plant grade of LNP Units 1 and 2.

2.4.3.4.6 Coincident Wind-Wave Activity

Information Submitted by the Applicant

The applicant stated that the maximum water surface elevation in Lake Rousseau during the PMF, which is estimated to be 9.1 m (29.7 ft) NAVD88, would be approximately 6.5 m (21.3 ft) below the nominal plant grade floor elevation of 15.5 m (51 ft) NAVD88. Based on this large difference, the applicant concluded that it is unlikely that a wind-wave activity coincident with the PMF would affect the safety-related facilities of the proposed LNP units.

NRC Staff's Technical Evaluation

The staff reviewed the methodology adopted by the applicant for the estimation of wind-induced waves and determined that the applicant did not consider wind-induced waves to be significant because the LNP site is located approximately 4.8 km (3 mi) from Lake Rousseau. After reviewing the applicant's responses to RAIs 02.04.03-05 and 02.04.03-06, the staff has determined that the applicant-estimated maximum water surface elevation in Lake Rousseau during a PMF event (9.1 m (29.7 ft) NAVD88) is acceptable. The maximum water surface elevation of 9.1 m (29.7 ft) NAVD88 in Lake Rousseau does not include wind-wave effects. Because the maximum stillwater elevation of 9.1 m (29.7 ft) NAVD88 in Lake Rousseau is more than 6.4 m (21 ft) below the nominal plant grade of LNP Units 1 and 2, the staff concluded that there is significant margin available between the stillwater elevation and the nominal plant grade. Wind-wave activity from a 2-year coincident wind is unlikely to exceed the available margin. Therefore, the staff concluded that a PMF in the Withlacoochee River Basin would not result in flooding at the LNP site.

The staff had not determined the maximum water surface elevation near the LNP site because the applicant's PMF analysis for the Withlacoochee River Basin was incomplete (see RAIs 02.04.03-05 and 02.04.03-06 above). Because of this issue, the determinations of the PMF water surface elevation and the design basis floodwater surface elevation at the LNP site were incomplete. Therefore, the staff considers RAIs 02.04.03-05 and 02.04.03-06 to be resolved.

2.4.3.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.3.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to PMF on streams and rivers, and that there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.3 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses COL Information Item 2.4-2.

2.4.4 Potential Dam Failures

2.4.4.1 *Introduction*

FSAR Section 2.4.4 of the LNP COL application addresses potential dam failures to ensure that any potential hazard to safety-related structures due to failure of onsite, upstream, and downstream water-control structures is considered in the plant design.

Section 2.4.4 of this SER presents a review of the specific areas related to dam failures. The specific areas of review are as follows: (1) flood waves resulting from severe dam breaching or failure, including those due to hydrologic failure as a result of overtopping for any reason, routed to the site and the resulting highest water surface elevation that may result in the flooding of SSCs important to safety; (2) successive failures of several dams in the path to the plant site caused by the failure of an upstream dam due to plausible reasons, such as a probable maximum flood, landslide-induced severe flood, earthquakes, or volcanic activity and the effect of the highest water surface elevation at the site under the cascading failure conditions; (3) dynamic effects of dam failure-induced flood waves on SSCs important to safety; (4) failure of a dam downstream of the plant site that may affect the availability of a safety-related water supply to the plant; (5) effects of sediment deposition or erosion during dam failure-induced flood waves that may result in blockage or loss of function of SSCs important to safety; (6) failure of onsite water-control or storage structures such as levees, dikes, and any engineered water storage facilities that are located above site grade and may induce flooding at the site; (7) the potential effects of seismic and non-seismic data on the postulated design bases and how they relate to dam failures in the vicinity of the site and the site region; and (8) any additional information requirements prescribed in the “Contents of Application” sections of the applicable subparts to 10 CFR Part 52.

2.4.4.2 *Summary of Application*

This section of the COL FSAR addresses the site-specific information about potential dam failures. The applicant addressed the information as follows:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.4-2

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 19 of the AP1000 DCD.

> Combined License applicants referencing the AP1000 design will address the following site-specific information on historical flooding and potential flooding factors, including the effects of local intense precipitation.

- Probable Maximum Flood on Streams and Rivers – Site-specific information that will be used to determine design basis flooding at the site. This information will include the probable maximum flood on streams and rivers.

- Dam Failures – Site-specific information on potential dam failures.

- Probable Maximum Surge and Seiche Flooding – Site-specific information on probable maximum surge and seiche flooding.

- Probable Maximum Tsunami Loading – Site-specific information on probable maximum tsunami loading.

- Flood Protection Requirements – Site-specific information on flood protection requirements or verification that flood protection is not required to meet the site parameter of flood level.

No further action is required for sites within the bounds of the site parameter for flood level.

This section of the SER relates to dam failures.

2.4.4.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification of floods, flood design considerations and potential dam failures, and the associated acceptance criteria, are described in Section 2.4.4 of NUREG-0800 (NRC 2007a).
The applicable regulatory requirements for identifying the effects of dam failures are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Appropriate sections of the following RGs are used by the staff for the identified acceptance criteria:

- RG 1.59, “Design Basis Floods for Nuclear Power Plants” (NRC 1977a), as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants" (NRC 1976b).

2.4.4.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.4 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the potential dam failure. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

The staff needs an accurate description of the assessment of the potential dam failures to perform its safety assessment. In RAI 2.4.4-1, the staff requested additional information regarding the applicant's process to determine the conceptual models for flood waves from severe breaching of upstream dams, domino-type or cascading failures of dams, dynamic effects on safety-related SSCs, loss of safety-related water supplies, sediment deposition and erosion, and failure of on-site water control or storage structures to ensure that the most conservative of plausible conceptual models has been identified.

In a letter dated June 15, 2009 (ML091680038), the applicant's response stated that the safety-related SSCs of LNP Units 1 and 2 are located in the Waccasassa River Basin, which does not have any water-control structures. Therefore, the applicant concluded that the LNP site would be unaffected by severe breaching of upstream dams. Because the nearest water-control structures, Inglis Dam and Spillway and Inglis Lock, are present in the adjoining Withlacoochee River Basin, the applicant analyzed the potential failure of these with a coincident high tide in the Gulf of Mexico. The applicant estimated that the maximum water surface elevation in the Lower Withlacoochee River due to the failure of the Inglis Dam during a PMF event would be approximately 8.2 m (27 ft) lower than the nominal plant grade floor elevation. The applicant did not analyze other water-control structures in the Withlacoochee River Basin upstream of the Inglis Dam because the topographic relief in the river basin is low. The applicant postulated that the flood wave caused by an upstream dam failure would spread in marshlands adjacent to the river channel and therefore would not affect Lake Rousseau or the LNP site.

The staff reviewed the applicant's response and determined that the applicant has adequately identified the dam breach scenarios that may affect the LNP site. However, there are two issues that the staff would independently check in order to verify the applicant's conclusion that upstream dam failures in the Withlacoochee River Basin would not affect the LNP site. The two issues are related to the effects of peaking of unit hydrographs and upstream dam failures on the water surface elevation of Lake Rousseau during a PMF event. These issues are described below.

2.4.4.4.1 Dam-Failure Permutations

Information Submitted by the Applicant

The applicant did not identify any dam-failure permutations. The applicant only postulated and analyzed the failure of the Inglis Dam. The applicant used the Froehlich (1995) method to estimate the peak flow from a postulated failure of the Inglis Dam. To estimate the peak flow, the applicant postulated that Lake Rousseau's storage and height of water at the time of failure would be at their respective maximums, 41,938,381 m^3 (34,000 ac-ft) and 9.4 m (30.7 ft). The applicant-estimated peak discharge from the postulated failure of Inglis Dam is 1,722 m^3/s (60,811 cfs). The applicant noted that in comparison, its estimate of maximum outflow from Lake Rousseau during the PMF event in the Withlacoochee River Basin is 1,720 m^3/s (60,755 cfs).

The applicant used the USACE HEC-RAS model to simulate a steady flow of 1,722 m^3/s (60,811 cfs) through a channel reach downstream of the Inglis Dam. The applicant selected a downstream boundary condition at the shoreline on the Gulf of Mexico equal to the 10 percent exceedance high tide. The applicant obtained a maximum water surface elevation of 7.53 m (24.72 ft) NGVD29. The applicant concluded that a postulated failure of the Inglis Dam would not result in a maximum water surface elevation exceeding 7.3 to 7.6 m (24 to 25 ft) NGVD29 downstream of the dam.

NRC Staff's Technical Evaluation

The staff requires information about all existing and proposed water retaining and water-control structures in the vicinity of the LNP site to ascertain that their possible effects are accounted for in the estimation of the design-basis flood. Because the applicant did not identify dams and water-control structures upstream of Lake Rousseau, in addition to the inflow hydrograph issues described in RAIs 02.04.03-05 and 02.04.03-06, the staff were not able to complete the review of dam failures and their potential effects on the LNP site. In RAI 2.4.4-2, the staff requested additional information related to all existing and proposed water retaining and water control structures both upstream and downstream relative to the LNP site location, including a justification of why failure of these structures would not affect flood elevations near the LNP site.

The applicant responded to the staff's RAI 02.04.04-02 in a letter dated June 15, 2009 (ML091680038). The applicant stated that it reviewed the USACE's National Inventory of Dams database to determine characteristics of dams in the Withlacoochee River Basin. The applicant listed 15 dams in the Withlacoochee River Basin with a total storage capacity of 271 million m^3 (219,650 ac-ft). The heights of these dams range from 3.7 to 16.8 m (12 to 55 ft).

The applicant stated that the difference between the operating pool elevation of Lake Rousseau and the nominal plant floor grade elevation is 7.3 m (24 ft). Because topographical relief in the Withlacoochee River Basin is low, the applicant concluded that floodwaters from a dam-failure event would spread out into marshlands located adjacent to the river channel and therefore not reach the LNP site.

The staff reviewed the applicant's response to RAI 02.04.04-02 and determined that the LNP nuclear island, which has SSCs important to safety, is not located in the Withlacoochee River Basin. The applicant has analyzed a postulated failure of the Inglis Dam but did not consider upstream dam failures. The applicant's reasoning for not considering upstream dam failures is that due to the low topographical relief in the Withlacoochee River Basin, floodwaters from an upstream dam-failure event would spread out into marshlands. The staff determined that the applicant has not shown, using observed data or simulations, that floodwaters in the Withlacoochee River Basin would indeed spread out into marshlands and not affect the water surface elevation in Lake Rousseau.

The staff independently assessed the effect of upstream dam failures in the Withlacoochee River Basin. The applicant identified 15 dams in the Withlacoochee River Basin, 13 of which are located upstream of Lake Rousseau. The applicant stated in response to RAI 02.04.04-02 that there are seven settling areas located in the southern part of the Withlacoochee River Basin, three of which have storage capacities exceeding 12.3 million m^3 (10,000 ac-ft). The applicant also stated that all the settling areas are hydrologically disconnected from the Withlacoochee River. The staff performed a search of the National Inventory of Dams database and found that the Saddle Creek settling areas are listed as privately owned earthen dams. Although the staff was able to find some references to settling areas created near the southern end of the Withlacoochee River Basin (SWFWMD 2009a), it was unable to verify whether these settling areas are hydrologically disconnected from the Withlacoochee River. Therefore, the staff included all 13 dams located upstream of Lake Rousseau in its analysis.

The staff independently determined the effects of upstream dam breaches using two scenarios that may affect water surface elevation in Lake Rousseau and downstream of the lake. The staff's two scenarios are: (1) the estimation of water surface elevation in Lake Rousseau because of failures of all upstream dams during the PMF event while the Inglis Dam remains intact and (2) the estimation of water surface elevation downstream of Lake Rousseau with failure of Inglis Dam coincident with the first scenario. The first scenario would result in the maximum water surface elevation in Lake Rousseau because the Inglis Dam would not fail and the second scenario would maximize the water surface elevation downstream of the Inglis Dam because Inglis Dam's failure would augment the discharge through Lake Rousseau postulated in the first scenario.

The staff assumed that the dams on Saddle Creek settling areas would fail simultaneously as a group and their peak discharges would arrive simultaneously at the outlet of the subbasin in which they are located. The staff also assumed that the Lake Tsala Apopka group of dams, Rufe Wysong Dam, Gant Lake Dam, and the Slush Pond Dam would fail as a group and their peak discharges would arrive at the outlet of the subbasin in which the Lake Tsala Apopka group of dams is located. Because Rufe Wysong Dam, Gant Lake Dam, and the Slush Pond Dam are located upstream of the Lake Tsala Apopka group of dams, the staff's assumption does not consider the attenuation and time lag in their discharges that would occur as the discharge flows downstream. Therefore, the staff's assumption is conservative and would result in greater peak discharges in the Withlacoochee River Basin downstream of the Lake Tsala Apopka group of dams.

The staff used the Froehlich (1995) approach to estimate the peak discharges from all dams using the data provided by the applicant in response to RAI 02.04.04-02. The staff independently verified these peak discharges, which are listed in Table 2.4.4-1. The staff estimated that the combined peak discharge of the dams on Saddle Creek settling area would be 6,524 m^3/s (230,388 cfs) and that for the Lake Tsala Apopka group of dams, Rufe Wysong Dam, Gant Lake Dam, and the Slush Pond Dam would be 3,329 m^3/s (117,546 cfs).

Table 2.4.4-1. Staff-Estimated Peak Discharges from Postulated Failures of Dams Upstream of Lake Rousseau

Dam Name	Maximum Storage (m^3 [ac-ft])	Height (m [ft])	Peak Discharge[1] (m^3/s [cfs])
Brogden Bridge - Lake Tsala Apopka[2]	36,634,409 (29,700)	5.2 (17)	795,1 (28,077.9)
Golf Course Bridge - Lake Tsala Apopka[2]	50,983,503 (41,333)	4.0 (13)	628.4 (22,194.3)
Structure 353 Bridge - Lake Tsala Apopka[2]	74,008,908 (60,000)	5.3 (17.5)	1,014.2 (35,815.1)
Slush Pond[2]	62,908 (51)	15.2 (50)	463.1 (16,353.1)
Gant Lake Dam[2]	651,278 (528)	3.7 (12)	157.2 (5,552.7)
Rufe Wysong Dam[2]	1,603,526 (1,300)	4.6 (15)	270.5 (9,552.4)
Saddle Creek Settling Area No. 1[3]	13,340,206 (10,815)	7.9 (26)	999.5 (35,297.9)
Saddle Creek Settling Area No. 2[3]	19,452,008 (15,770)	7.3 (24)	1,011.6 (35,724.5)
Saddle Creek Settling Area No. 3[3]	4,576,217 (3,710)	5.8 (19)	494.1 (17,448.7)

To create a discharge hydrograph for the combined discharge of the two groups of dams, the staff assumed that all of the storage in the dams within a group would be released during their failure. The staff assumed that the hydrographs would have a triangular shape with a peak discharge equal to the combined peak discharge of the group.

The staff used the Withlacoochee River Basin HEC-HMS model provided by the applicant and modified it to include the two conservatively estimated discharge hydrographs resulting from the respective failures of the two groups of dams in the model at the appropriate locations. The staff simulated the PMF scenario, which now includes conservatively estimated upstream dam-failure hydrographs. The staff's HEC-HMS simulation resulted in a peak outflow discharge of 1,751 m^3/s (61,851 cfs) and a maximum water surface elevation of 9.1 m (29.7 ft) NGVD29 in Lake Rousseau. Therefore, the staff concluded that for the staff's first scenario listed above, the LNP site would be safe from flooding because the plant grade elevation is more than 6.1 m

(20 ft) above the maximum water surface elevation in Lake Rousseau caused by upstream dam failures coincident with the PMF event.

For the staff's second scenario, the staff concluded that the maximum water surface elevation in Lake Rousseau during upstream dam failures coincident with a PMF event in the Withlacoochee River Basin would not exceed 9.1 m (30 ft) NGVD29. Therefore, the applicant's estimate of peak discharge during a postulated failure of the Inglis Dam is conservative because the applicant used a water height of 9.4 m (30.7 ft). The peak discharge of 1,751 m^3/s (61,851 cfs) from Lake Rousseau as estimated by the staff is greater than that estimated by the applicant (1,716 m^3/s [60,597 cfs]) by about 2 percent. The staff's independent assessment described below also showed that increasing the applicant-estimated peak discharge from Lake Rousseau by 50 percent did not result in an appreciable rise in the maximum water surface elevation downstream of Lake Rousseau. To estimate the water surface elevation below Lake Rousseau for the staff's second scenario (failure of Inglis Dam coincident with PMF in Withlacoochee River Basin and failure of upstream dams), the staff conservatively assumed that the discharge from Lake Rousseu would be a combination of peak discharge estimated for the PMF event coincident with upstream dam failures and the peak discharge because of breach of Inglis Dam. Because the staff estimated that peak discharge from Lake Rousseau during the PMF event coincident with upstream dam failures is greater than the peak discharge from the single failure of Inglis Dam, the staff conservatively estimated the combined discharge by doubling the staff-estimated peak discharge from for the PMF event coincident with upstream dam failures. Therefore, the staff-estimated peak discharge for the second scenario is 3,502 m^3/s (123,702 cfs).

The staff performed a steady-state simulation using the HEC-RAS model provided by the applicant with an input discharge of 3,502 m^3/s (123,702 cfs). The staff determined that the maximum water surface elevation below Lake Rousseau for the second scenario would be approximately 9.7 m (31.8 ft) NGVD29. Therefore, the staff concluded that failure of Inglis Dam during the PMF event and coincident upstream dam failures would not result in a flood hazard at the LNP site.

2.4.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

Information Submitted by the Applicant

The applicant did not perform an unsteady flow analysis of potential dam failures. The peak discharge following the failure of the Inglis Dam was used in a steady flow simulation to estimate water surface elevation downstream of the Inglis Dam.

NRC Staff's Technical Evaluation

The staff reviewed the methodology adopted by the applicant in its estimation of design basis floodwater surface elevations. To verify the conservativeness of the applicant's approach, the staff issued **RAI 02.04.04-03,** which states the following:

> To meet the requirements of GDC 2, 10 CFR 52.17, 10 CFR Part 100, and 10 CFR 100.23(d), an appropriate configuration of the cascade of dam failures and its potential to produce the largest flood adjacent to the plant site is needed. Flood waves produced by postulated dam failure scenarios should be routed to the proposed plant site to conservatively estimate the most severe floodwater surface elevation that may affect SSCs important to safety. Please clarify the steady flow methodology for analysis of the dam break-induced flood and to justify why the estimated flood water surface elevations are conservative.

The applicant responded to the staff's RAI 02.04.04-03 in a letter dated June 15, 2009 (ML091680038). The applicant stated that its steady-state analysis of the postulated Inglis Dam and Inglis Lock failure used a downstream water surface elevation specified by a 10 percent exceedance tide. The applicant stated that flood discharge and water surface elevations estimated by a steady-state approach are overestimated for a flow event that is transient. The staff's confirmatory analyses agree with the applicant's explanation. Therefore, the staff concludes that the steady-state simulation used by the applicant would result in a conservative estimate of the floodwater surface elevation.

2.4.4.4.3 Water Level at the Plant Site

Information Submitted by the Applicant

The applicant used the USACE HEC-RAS computer program to estimate water surface elevations downstream of the Inglis Dam after the failure of the dam. The applicant estimated the cross sections of the floodplain from downstream of the Inglis Dam to the Gulf of Mexico using USGS digital terrain data (Figure 2.4.4-1, adapted from FSAR Revision 0 Figure 2.4.4-201). The applicant estimated that the maximum water surface elevation downstream of the Inglis Dam due to its failure would be 7.53 m (24.72 ft) NGVD29. The applicant concluded that the LNP site would not be adversely affected by this flood.

LNP COL 2.4-2

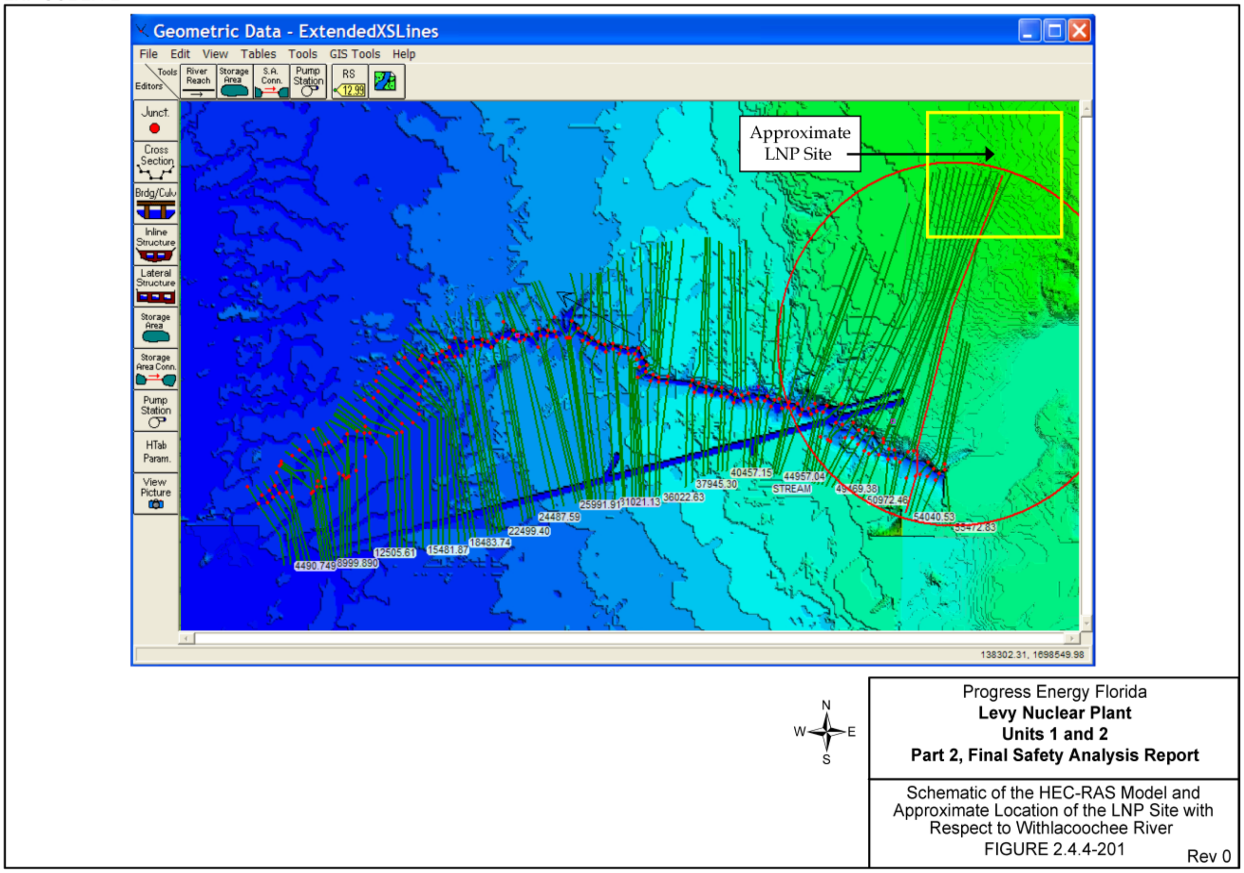


Figure 2.4.4-1. The Cross Sections Used in the HEC-RAS Simulation by the Applicant

<u>NRC Staff's Technical Evaluation</u>

The staff performed an independent analysis to estimate the sensitivity of floodwater surface elevations with respect to the applicant-selected parameters of the dam-failure scenario. The staff considered two cases: (1) a 50 percent increase in the peak discharge used in the applicant's HEC-RAS steady-state simulation and (2) an increase in Manning's n by 50 percent. The staff found that the maximum water surface elevation predicted by HEC-RAS is only minimally sensitive to the altered parameters. The maximum water surface elevation predicted by HEC-RAS for the two sensitivity simulations was 7.9 m (26 ft) NGVD29 compared to the applicant's estimate of 7.53 m (24.72 ft) NGVD29. Therefore, the staff concluded that it is unlikely that the LNP site could be inundated by a dam breach event postulated by the applicant.

The staff has independently assessed two issues in order to verify the applicant's conclusion that upstream dam failures in the Withlacoochee River Basin would not affect the LNP site. The first of these issues was described in RAIs 02.04.03-05 and 02.04.03-06 and addressed peaking of the unit hydrographs used in the PMF simulations. It is plausible that the inflow hydrograph into Lake Rousseau during the PMF would be more severe if peaked unit hydrographs were used in the PMF simulations, which may increase the discharge after the postulated breach of the Inglis Dam. The applicant addressed this issue in response to RAI 02.04.03-06. As stated in Section 2.4.3 of this SER, based on the applicant's response to the staff's RAI 02.04.03-06, the staff concluded that the applicant has used appropriate and conservative methods in the estimation of the PMF in the Withlacoochee River Basin upstream of the Inglis Dam. The second issue with regard to the effect of upstream dam failures on water surface elevations in Lake Rousseau stems from the plausible consideration that upstream dam failures could occur during PMF conditions in the Withlacoochee River Basin. The staff independently assessed the effects of increased water level in Lake Rousseau, as described in the applicant's responses to RAIs 02.04.03-05 and 02.04.03-06. The staff's independent assessment of dam failures in the Withlacoochee River Basin upstream of Lake Rousseau is described in Section 2.4.4.4.1 of this SER.

The staff performed an independent assessment of dam failures in the Withlacoochee River Basin upstream of Lake Rousseau after the applicant responded to staff's RAIs 02.04.03-05, 02.04.03-06, and 02.04.04-02. The staff's independent assessment is described in Section 2.4.4.4.1 of this SER. Based on its independent assessment, the staff concluded that failures of dams in the Withlacoochee River Basin upstream of Lake Rousseau would not result in flooding of the LNP site. The staff also concluded that failure of Inglis Dam coincident with a PMF event and upstream dam failures would not result in appreciable increase water surface elevations downstream of the dam to affect the LNP site. Therefore, the staff considers RAIs 02.04.03-05, 02.04.03-06, and 02.04.04-02 to be resolved.

2.4.4.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.4.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to potential dam failures, and that no outstanding information is expected to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.4 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses part of COL information item 2.4-2.

2.4.5 Probable Maximum Surge and Seiche Flooding

2.4.5.1 *Introduction*

FSAR Section 2.4.5 of the LNP COL application addresses the probable maximum surge and seiche (PMSS) flooding to ensure that any potential hazard to the safety-related SSCs at the proposed site has been considered in compliance with the Commission's regulations.

Section 2.4.5 of this SER presents evaluation of the following topics based on data provided by the applicant in the FSAR and information available from other sources: (1) probable maximum hurricane (PMH) that causes the probable maximum surge as it approaches the site along a critical path at an optimum rate of movement; (2) probable maximum wind storm (PMWS) from a hypothetical extratropical cyclone or a moving squall line that approaches the site along a critical path at an optimum rate of movement; (3) a seiche near the site, and the potential for seiche wave oscillations at the natural periodicity of a waterbody that may affect floodwater surface elevations near the site or cause a low water surface elevation affecting safety-related water supplies; (4) wind-induced wave run-up under PMH or PMWS winds; (5) effects of sediment erosion and deposition during a storm surge and seiche-induced waves that may result in blockage or loss of function of SSCs important to safety; (6) the potential effects of seismic and non-seismic information about the postulated design bases and how they relate to a surge and seiche in the vicinity of the site and the site region; (7) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.5.2 *Summary of Application*

This section of the COL FSAR addresses the site-specific information about PMSS flooding in terms of impacts on structures and water supply. The applicant addressed these issues as follows:

AP1000 COL Information Item

- LNP COL 2.4-2

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 19 of the AP1000 DCD.

> Combined License applicants referencing the AP1000 design will address the following site-specific information on historical flooding and potential flooding factors, including the effects of local intense precipitation.
>
> - Probable Maximum Flood on Streams and Rivers – Site-specific information that will be used to determine design basis flooding at the site. This information will include the probable maximum flood on streams and rivers.
>
> - Dam Failures – Site-specific information on potential dam failures.

- Probable Maximum Surge and Seiche Flooding – Site-specific information on probable maximum surge and seiche flooding.

- Probable Maximum Tsunami Loading – Site-specific information on probable maximum tsunami loading.

- Flood Protection Requirements – Site-specific information on flood protection requirements or verification that flood protection is not required to meet the site parameter of flood level.

No further action if required for sites within the bounds of the site parameter for flood level.

2.4.5.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification of floods, flood design considerations and potential dam failures, and the associated acceptance criteria, are described in Section 2.4.5 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying the effects of dam failures are:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d) sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Appropriate sections of the following RGs are used by the staff for the identified acceptance criteria:

- RG 1.59, "Design Basis Floods for Nuclear Power Plants" (NRC 1977a), as supplemented by best current practices; and

- RG 1.102, "Flood Protection for Nuclear Power Plants" (NRC 1976b).

2.4.5.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.5 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the probable maximum surge and seiche flooding. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

2.4.5.4.1 Probable Maximum Winds and Associated Meteorological Parameters

Information Submitted by the Applicant

The applicant stated that between the years 1851 and 2006, northwest Florida was struck by 57 hurricanes. Fourteen of these hurricanes were classified as major hurricanes but none were of Category 4 or 5.

The applicant estimated the meteorological parameters of the PMH from NOAA NWS Report 23. The applicant-estimated PMH parameters are listed in Table 2.4.5-1.

Table 2.4.5-1. Applicant-Estimated PMH Parameters

Parameter	Minimum Value	Maximum Value	Unit
Central pressure	88.9 (889)	89.1 (891)	kPa (millibar)
Peripheral pressure	102 (1,020)	102 (1,020)	kPa (millibar)
Radius of maximum winds	12.4 (6.7)	41.3 (22.3)	km (nautical mile)
Forward speed	25.7 (16)	37 (23)	km/hr (mi/hr)
Maximum wind speed	251 (156)	252.7 (157)	km/hr (mi/hr)
Track direction	200	245	degree from north

The applicant estimated the 10 percent exceedance high spring tide of 1.3 m (4.3 ft) mean low water from RG 1.59 (NRC 1977a). The applicant reported a maximum astronomical tide of 1.5 m (4.9 ft) mean lower-low water based on tide data at Cedar Key, Florida.

NRC Staff's Technical Evaluation

An accurate description of the assessment of PMSS events at the LNP site is needed for the staff to perform its safety assessment. To resolve inconsistencies observed in the information presented by the applicant with regard to observed hurricanes, tropical storms, and tropical depressions, staff issued **RAI 02.04.05-01**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge, are needed. The PMH, as defined by NOAA NWS Report 23, should be estimated for coastal locations that may be exposed to these events. In the FSAR text, it is stated that FSAR Table 2.4.5-201 contains a list of hurricanes that came within 80.5 km (50 mi) of the LNP site during 1867–2004. The table contains a list of events that includes hurricanes, tropical storms, and tropical depressions. Please resolve this inconsistency.

The applicant responded to the staff's RAI 02.04.05-01 in a letter dated July 20, 2009 (ML092030128). The applicant agreed with the staff's observation regarding FSAR Table 2.4.5-201 and updated that table to include only a list of recorded hurricanes.

In RAI 2.4.5-2, the staff requested additional information related the applicant's use of Hsu's empirical equation for the estimation of PMH storm surge and why the applicant considered the estimated coastal storm surge elevations under PMH conditions to be conservative.

The applicant responded to the staff's RAI 02.04.05-02 in a letter dated July 20, 2009 (ML092030128). The applicant stated that Hsu's method (Hsu et al. 2006), which uses three key pieces of information—minimum sea level pressure, shoaling factor, and correction factor for storm motion—has been validated using data from recent hurricanes, including Katrina and Rita. The applicant used parameters of a PMH storm to estimate the PMSS at the coastline and compared it to the coastal storm surge elevations given in RG 1.59 (NRC 1977a). The applicant-estimated coastal storm surge including the 10 percent exceedance high tide using Hsu's method (Hsu et al. 2006) was slightly higher than that obtained by converting the value specified in RG 1.59 (NRC 1977a) to the same datum. The applicant concluded therefore, that Hsu's method (Hsu et al. 2006) is conservative.

The staff reviewed the applicant's response to RAI 02.04.05-02 and calculations to determine that Hsu's empirical method (Hsu et al. 2006) produced a higher storm surge estimate that that specified in RG 1.59 (NRC 1977a) at the coastline near the LNP site. Therefore, the staff agrees with the applicant that Hsu's empirical method (Hsu et al. 2006) is conservative insofar as it is used to estimate coastal storm surge near the LNP site.

2.4.5.4.2 Surge and Seiche Water Levels

Information Submitted by the Applicant

The applicant used three approaches for estimating the PMH storm surge at the LNP site. These methods are based on (1) guidance in RG 1.59 (NRC 1977a), (2) results obtained by NOAA NWS using its Sea, Lake, and Overland Surge from Hurricanes (SLOSH) model for several combinations of hurricane parameters, and (3) correlating the SLOSH estimates with an empirical equation.

Storm Surge Estimate from Regulatory Guide 1.59

The applicant assumed that the estimates of storm surge at Crystal River provided in Appendix C of RG 1.59 (NRC 1977a) are applicable for the LNP site because of the proximity of the site to this location. The applicant obtained the following PMH storm surge parameters on the open coast near Crystal River from RG 1.59 (NRC 1977a):

Wind setup	8.1 m (26.55 ft)
Pressure setup	0.8 m (2.65 ft)
Initial rise	0.2 m (0.6 ft)
10 percent exceedance high tide	1.3 m (4.3 ft) MLW
Total surge	10.4 m (34.1 ft) MLW

Storm Surge Estimate from NOAA NWS SLOSH Runs

The applicant stated that SLOSH model results are generally accurate to approximately 20 percent of the computed value. The applicant chose four coastal points near the LNP site and extracted the maximum of the maximum envelope of water (MOM) values from NOAA NWS pre-computed SLOSH model runs for hurricanes of Categories 1 through 5. The applicant also obtained the MOM values for the towns of Yankeetown and Inglis and for the location of the LNP site. The SLOSH model MOM scenarios predicted that the LNP site would be dry from storm surge caused by hurricanes of Categories 1 through 5.

Storm Surge Estimate for the PMH Using Hsu's Empirical Method

The applicant used an empirical equation proposed by Hsu et al. (2006) to estimate the open coast PMH storm surge. The equation uses two empirical coefficients, one called the shoaling factor and the other the storm motion factor, along with a minimum sea-level pressure for the hurricane. The applicant estimated the shoaling coefficient using the location of the coast near the LNP site, specifically the Cedar Key NOAA gauge site, along with a nomograph provided by Hsu et al. (2006). The storm motion factor was estimated using PHM storm track parameters, forward speed, and track direction (see Table 2.4.5-1), along with a nomograph provided by Hsu et al. (2006). The applicant reported that the maximum value of the storm motion factor was estimated to be 0.7.

The applicant estimated the storm surge heights induced by hurricanes of Categories 1 through 5 at the coast using Hsu's method (Hsu et al. 2006) and compared them to the average of the previously selected four coastal points' storm surge estimated by the SLOSH model. The applicant concluded that because storm surges estimated by Hsu's method (Hsu et al. 2006) were consistently higher than those from the SLOSH model, results obtained from Hsu's method (Hsu et al. 2006) were conservative.

The applicant obtained a relationship between inland storm surge heights and the coastal storm surge heights from NOAA NWS pre-computed SLOSH model runs for two locations: Yankeetown and Inglis. A similar relationship for storm surge at the LNP site could not be obtained because the LNP site location was dry in all SLOSH model runs. The applicant concluded that these two relationships, for Yankeetown and Inglis, could be used to estimate the storm surge height at the inland location if the storm surge height at the Gulf coast was known, irrespective of the intensity of the hurricane.

The applicant proposed that the storm surge at the LNP site be obtained from an extrapolation relationship based on the storm surge heights at Yankeetown and Inglis and the corresponding distances of the three locations from the Gulf coast. Using this relationship, the applicant estimated the storm surge height at the LNP site for hurricanes of Categories 1 through 5. All of these storm surges heights were reported as “(dry)” in FSAR Revision 0 Table 2.4.5-214.

The applicant performed a set of estimation of storm surge at the LNP site using 1000 randomly selected combinations of PMH parameters. The applicant did not provide any detail about how storm surge at the LNP site was obtained from these sets of PMH parameters. The maximum applicant-estimated stillwater storm surge at the LNP site was 12.60 m (41.33 ft).

The applicant did not consider seiches in Lake Rousseau as the controlling influence and stated that the potential for flooding at the site due to seiches in Lake Rousseau is insignificant.

NRC Staff’s Technical Evaluation

The NRC staff reviewed the analysis and data provided by the applicant. To obtain clarification on the conversion of datums and tabular presentation of data used in the applicant’s analysis, the staff issued **RAI 02.04.05-03**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH should be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. Please clarify the details of how the conversion from MSL to NGVD29 was made and provide details of how the Hsu method storm surge heights in FSAR Table 2.4.5-213 were obtained. Please clarify why the table is titled "PMH Analysis for the LNP Site," since it appears that the values reported in this table are for storm surges for hurricanes of categories 1 through 5 and not for the PMH.

The applicant responded to the staff’s RAI 02.04.05-03 in a letter dated July 20, 2009 (ML092030128). The applicant stated that the Cedar Key tidal datum was used to convert water surface elevation from mean sea level to NGVD29 and NAVD88 datums. The applicant used the NOAA VERTCON tool to convert between NGVD29 and NAVD88 datums. The staff determined in its independent review that the Cedar Key NOAA tide gauge is located closest to the LNP site and therefore is the most appropriate location to use for antecedent tidal elevations.

The applicant stated that storm surge water surface elevations reported in FSAR Table 2.4.5-213 were obtained using Hsu’s empirical equation (Hsu et al. 2006) along with parameters for hurricanes of Category 1 through 5 listed in FSAR Table 2.4.5-205, with the mean of the atmospheric pressure range used for each hurricane category in the equation. The staff reviewed Hsu's methodology (Hsu et al. 2006) along with the parameters listed in FSAR Table 2.4.5-205 and determined that the applicant has adequately used the empirical method.

The applicant stated that FSAR Table 2.4.5-213 was labeled "PMH Analysis for the LNP Site" because it represents on step in the process of estimating the PMSS at the LNP site. The applicant stated that the title of the table would be revised for clarity. To resolve inconsistencies in the application of the SLOSH model as presented in the FSAR, the staff issued **RAI 02.04.05-04**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and10 CFR Part 100, an estimate of wind-induced wave runup under PMH winds is needed. The controlling flood water surface elevations are estimated based on the combination of appropriate ambient water surface elevations, critical storm surge or seiche water surface elevations, and coincident wind-wave action as described in ANSI/ANS-2.8-1992.
>
> (1) The applicant stated in FSAR Revision 0, Section 2.4.5.2.3 page 2.4-37: "Since the datum used in the SLOSH model is NGVD, formerly known as the Sea Level Datum of 1929, an astronomical tide level above NGVD29 would add additional height to the values computed by the SLOSH model. Thus, the SLOSH model accounts for astronomical tides." Jelesnianski et al. (1992) clearly state that astronomical tide is ignored by the SLOSH model except for its superposition onto the computed surge. The applicant's statement conveys a broader interpretation of the capabilities of the SLOSH model in how it incorporates the effect of astronomical tide in surge computations.
>
> (2) The applicant stated in FSAR Revision 0, Section 2.4.5.2.3 page 2.4-37: "Generally, waves do not add significantly to the total area flooded by storm surge and can usually be ignored." The applicant also stated in FSAR Revision 0, Section 2.4.5.3.1 page 2.4-41: "As mentioned in FSAR Subsection 2.4.5.2.3, the SLOSH model does not include the additional heights generated by wind-driven waves on top of the stillwater storm surge. Therefore, wind-driven wave height needs to be determined." While the first statement may be true inasmuch as the area of inundation is concerned, it gives an impression that wind waves on top of storm surge stillwater elevation may be ignored, which is not the case, as stated by the second quote.
>
> Please resolve these inconsistencies, or explain why your statements are sufficient.

The applicant responded to the staff's RAI 02.04.05-04 in a letter dated July 20, 2009 (ML092030128). The applicant stated that the SLOSH model accounts for tides by specifying the initial tide level. The applicant stated that the SLOSH model results presented in FSAR Tables 2.4.5-206 through 2.4.5-209 used an initial tidal elevation of 0.8 m (2.5 ft) NGVD29, whereas the 10 percent exceedance tide for Cedar Key tidal gauge is 0.6 m (2.01 ft) NGVD29.

Therefore, the applicant concluded that its PMH analysis is based on a conservative estimate of the initial tidal elevation. The staff reviewed the applicant response and its calculation package to determine whether the initial tidal elevation is more conservative than the recommended 10 percent exceedance tide. Therefore, the staff determined that the applicant's PMSS estimates used a conservative value for initial tidal elevation.

The applicant stated that for clarity and to be more specific to site conditions, the statement "generally, waves do not add significantly to the total area flooded by storm surge and can usually be ignored" would be removed from the FSAR. The staff determined that the removal of the aforementioned phrase would clarify the contribution of wind driven waves to storm surge. The staff considers RAI 02.04.05-04 to be resolved.

To obtain clarification on the hydrodynamic basis of the analysis presented by the staff issued **RAI 02.04.05-05**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of the probable maximum hurricane (PMH) and the probable maximum storm surge are needed. The storm surge induced by the PMH should be estimated as recommended by Regulatory Guide 1.59, supplemented by current best practices. Please clarify and justify the hydrodynamic basis for the extrapolation equation, FSAR Revision 0 Equation 2.4.5-5, used for estimation of storm surge at the LNP site.

The applicant responded to the staff's RAI 02.04.05-05 in a letter dated July 20, 2009 (ML092030128). The applicant provided an explanation of how three methods, based on RG 1.59 (NRC 1977a), NOAA pre-computed SLOSH model simulations for hurricanes of Category 1 through 5, and Hsu's empirical approach (Hsu et al. 2006), were used in the FSAR. The applicant stated that the mechanism of propagation of waves and consequent flooding of inland locations is based on the SLOSH model pre-computed results. The applicant stated that extrapolation of the SLOSH model pre-computed results to predict the PMSS at the LNP site is based on hydrodynamics of the model itself.

The staff disagreed with the applicant's assessment because it used an extrapolation technique. Coastal hydrodynamics, especially the interaction of storm surge with inland topography is a highly complex and nonlinear process. The staff disagreed that the extrapolation procedure used by the applicant can accurately be used to predict the storm surge resulting from a PMH by only using a few points in the modeling domain. The staff also determined that a technically sound and demonstrably conservative approach should be used to estimate the PMSS at the LNP site. To resolve this pending issue, the staff drafted **RAI 02.04.05-09**, which states:

> In response to the staff's RAI 2.4.5-05, the applicant stated that the extrapolation equation that was used to estimate PMSS at the LNP site is based on National Oceanic and Atmospheric Administration National Weather Service's Sea, Lake and Overland Surges from Hurricanes (SLOSH) modeling results for hurricanes of Categories 1 through 5 in the Gulf of Mexico near the LNP site. Through independent confirmatory analysis, the staff determined that the Probable Maximum Storm Surge (PMSS) water surface elevations obtained by using the

> extrapolation procedure described by the applicant may be conservative, but is not technically valid because there is no hydrodynamic basis that captures the complex interaction of the storm surge and inland topography within the equation.
>
> Provide the following information: (a) an analysis of the PMSS event using a technically sound and conservative approach such as those predicted by a storm surge model (e.g., SLOSH) with input from appropriate Probable Maximum Hurricane scenarios, (b) an estimate of sea level rise accounting for current climatic predictions, and (c) if factored into the PMSS analysis (i.e., application of margins), a detailed description of the process for determining uncertainty estimations.

The applicant's responses to RAIs 02.04.05-10 and 02.04.05-11 described below, document the applicant's use of the SLOSH model to simulate PMH conditions directly as opposed to extrapolating from pre-existing Category 1 through 5 results. Because the applicant no longer relies on pre-computed SLOSH model scenarios for hurricanes of Categories 1 through 5, the portion of the RAI 02.04.05-05 related to the extrapolation method used before is obsolete.

To ascertain whether the applicant has considered other mechanisms in addition to surge in the determination of flooding at the site, the staff issued **RAI 02.04.05-06**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, estimates of seiche and resonance in waterbodies induced by meteorological causes, tsunamis, and seismic causes are needed. Please address the possibility of seiches of meteorological and seismic origin in Lake Rousseau; including, the possibility of resonance in Lake Rousseau that may amplify any potential seiche activity.

The applicant responded to the staff's RAI 02.04.05-06 in a letter dated July 20, 2009 (ML092030128). The applicant stated that Lake Rousseau is located approximately 4.8 km (3 mi) south of the LNP site and its operating pool elevation is maintained more than 6.1 m (20 ft) below the nominal plant grade floor elevation of safety-related structures to be built at the LNP site. Because of the significant difference in LNP nominal plant grade floor elevation and the operating pool elevation of Lake Rousseau and because of limited fetch due to the long and narrow shape of the lake, the applicant concluded that the possibility of a meteorologically induced seiche affecting LNP safety-related SSCs is insignificant. The applicant compared the runup and run-in induced by seismically generated tsunamis in the Gulf of Mexico—5.7 m (18.6 ft) and 0.89 km (0.55 mi), respectively—with the elevation and location of the LNP site and concluded that a seismically generated seiche would not affect the site. The applicant also stated that the possibility of resonance in Lake Rousseau due to a seismic event is insignificant.

The staff agrees with the applicant that a significant margin, greater than 6.1 m (20 ft), exists between the operating pool elevation of Lake Rousseau and the nominal plant grade floor elevation of safety-related SSCs. The staff reviewed the characteristics of Lake Rousseau and determined that it is a shallow lake, with an average depth of less than 3 m (10 ft). Also, because the lake is narrow and long in the east-west direction and the LNP site is located to its north, there is limited fetch available for waves to develop. Because of these characteristics, the staff determined that waves set up in Lake Rousseau would be limited by fetch and by water depth. The USACE Coastal Engineering Manual (CEM) (Scheffner 2008) suggests that waves are limited to 0.6 times water depth. The staff determined, therefore, that waves set up under most extreme meteorological conditions would not exceed approximately 1.8 m (6 ft) in height. Because the nominal plant grade floor elevation of safety-related SSCs at the LNP site is located more than 6.1 m (20 ft) above the operating pool elevation of Lake Rousseau, the staff concluded that meteorologically or seismically induced waves setup in the lake would not adversely affect the plant.

To ascertain that the applicant has considered all plausible PMH scenarios and used appropriate initial and boundary conditions in the analysis of surge staff issued **RAI 02.04.05-10**, which states:

> In RAI 2.4.5-09 (RAI ID 4629, Question 17567), the staff requested the applicant to provide the following information: (a) an analysis of the probable maximum storm surge (PMSS) event using a technically sound and conservative approach such as that predicted by a storm surge model (e.g., Sea, Lake, and Overland Surges from Hurricanes [SLOSH]) with input from appropriate Probable Maximum Hurricane (PMH) scenarios, (b) an estimate of sea level rise accounting for current climatic predictions, and (c) if factored into the PMSS analysis (i.e., application of margins), a detailed description of the process for determining uncertainty estimations. The applicant's response, dated June 18, 2010, does not appear to describe an estimation of PMSS at and near the LNP site using PMH scenarios input into a currently accepted hydrodynamic storm surge model. NRC requests that the applicant:
>
> (1) utilize a set of plausible PMH scenarios consistent with National Oceanic and Atmospheric Administration (NOAA) National Weather Service (NWS) Report 23 (NWS 23)[11] as input to a currently accepted storm surge model (such as SLOSH)
>
> (2) use initial open-water conditions that are consistent with current understanding of long-term sea-level rise and are valid for the life of the proposed plant
>
> (3) provide estimates of coincident wind-wave runup
>
> (4) maps of highest PMSS water surface elevation at and near the LNP site, and

[11] Schwerdt et al., 1979.

(5) provide updates to FSAR Section 2.4.5 including descriptions of data, methods, model setup, PHM scenarios and how they are consistent with NWS 23, treatment of uncertainty in the analysis, and available margins.

The applicant responded to the staff's RAI 02.04.05-10 in a letter dated January 27, 2011 (ML110340018). The applicant stated that it performed a confirmatory analysis using SLOSH Version 3.95 for the estimation of the PMH surge elevation at the LNP site. The applicant used the Cedar Key Basin for the analysis. The applicant selected PMH parameters based on NWS Report 23. The applicant determined the PMH antecedent water levels including a 10 percent exceedance spring high tide elevation of 0.98 m (3.23 ft) NAVD88 and a 100-year sea level rise of 0.18 m (0.59 ft) for a combined antecedent initial water level of 1.16 m (3.82 ft) NAVD88. The applicant simulated 576 preliminary cases using the SLOSH model, which varied in terms of landfall location, radius to maximum winds, forward speed, and track direction. The applicant examined the preliminary results and selected the case that yielded the highest water level. Based on this case, the applicant developed a refined and simulated a collection of new SLOSH cases to more precisely determine the conditions leading to the highest water elevation associated with the PMH. The applicant finally determined that a PMH with a radius to maximum winds of 41.8 km (26 mi), a forwards speed of 37 km/hr (23 mph) coming from 225 degree clockwise from north, yielded a surge at the LNP site of 14.5 m (47.7 ft) NAVD88 where the ground level is about 12.8 m (42 ft) (no datum given). The applicant determined that PMH wave setup at the LNP is 0.18 m (0.6 ft) and the wave runup is 0.45 m (1.48 ft) yielding a PMSS of 15.17 m (49.78 ft) NAVD88 (14.54 m (47.70 ft NAVD88) + 0.18 m (0.6 ft) + 0.45 m (1.48 ft)). The applicant reasoned that in the analysis described in the RAI response yielded a PMSS (15.17 m (49.78 ft) NAVD88) that closely corresponded with that previously described in the FSAR (15.09 m (49.52 ft) NAVD88), that the value presented in the FSAR would be used as the characteristic PMH flood elevation at the site.

The staff reviewed the applicant's approach to estimation of the initial water elevation for a hydrodynamic storm surge model using tidal data presented in RG 1.59 (NRC 1977a) for the Cedar Key tide gauge, and NOAA's description of predicted tides. The staff determined that NOAA estimates harmonic constants at reference tide stations that are used to predict the harmonic component of tidal variations at the reference stations. Observed tide water levels also include the effects of wind-wave activity and initial rise. Both of these additional components manifest as random variations added to the harmonic component of the tidal variations. Because these random variations are independent of the harmonic forcings (mainly gravitational forces of the sun and the moon) and therefore can occur at any time, there is no assurance the "high" random variations of tides would be in phase with the highs of the predicted tides. Therefore, estimating the 10 percent exceedance tide from raw tide water level observations can result in the underestimation of the initial water level (represented by 10 percent exceedance of predicted tides plus initial rise). RG 1.59 (NRC 1977a) does not describe how the initial rise reported for various locations in Appendix C of the guide was estimated. The staff concluded that the applicant had not provided sufficient information. Therefore, the staff issued **RAI 02.04.05-11**, which states:

In RAI 2.4.5-10, the staff requested the applicant to provide supplemental information; the staff stated that the applicant must (1) use a set of plausible probable maximum hurricane (PMH) scenarios consistent with the National Oceanic and Atmospheric (NOAA) National Weather Service (NWS) Report 23 (NWS 23) as input to a currently accepted storm surge model (such as NWS Sea, Lake, and Overland Surges from Hurricanes [SLOSH]), (2) use initial open-water conditions that are consistent with current understanding of long-term sea-level rise and are valid for the life of the proposed plants, (3) provide estimates of coincident wind-wave runup, (4) provide maps of highest probable maximum storm surge (PMSS) water surface elevation at and near the LNP sites, and (5) provide updates to FSAR Section 2.4.5, including descriptions of data, methods, model setup, PMH scenarios and how they are consistent with NWS 23, treatment of uncertainty in the analysis, and available margins.

The applicant responded to RAI 2.4.5-10 on January 27, 2011. The staff's review of the applicant's response to RAI 2.4.5-10 has raised the following issues:

(1) Regulatory Guide (RG) 1.59 recommends that the following components of PMSS be estimated: (a) probable maximum surge (wind and pressure setups), (b) 10 percent exceedance tide, and (c) initial rise (forerunner or sea-level anomaly). The wind wave runup also needs to be added to obtain the PMSS. The applicant did not use an initial rise in its SLOSH simulations. RG 1.59 recommends an initial rise of 0.6 ft for Crystal River, FL. Because the value of initial water surface can have nonlinear effects on SLOSH predictions, 10 percent exceedance tide, initial rise, and long-term sea level rise should be combined to specify the initial water surface in SLOSH for simulation of the PMH scenarios.

    In a subsequent teleconference, the applicant stated its interpretation of RG 1.59 recommendations. The applicant stated that RG 1.59 recommends use of initial rise as an additional component of the initial water level if the 10 percent exceedance tide is estimated from predicted tides. The applicant stated that use of initial rise is not necessary because its approach used observations of tidal water levels that already contain the effects of initial rise.

(2) The applicant has not used the US Army Corps of Engineers Coastal Engineering Manual (CEM) for estimation of coincident wind wave activity. The CEM approach is recommended in SRP 2.4.5 as the currently accepted practice. The applicant did not provide justification why it used another approach. In a subsequent teleconference, the applicant stated that they did in fact use the CEM approach to estimate wind wave activity although this fact was not clearly stated in the response to RAI 2.4.5-10.

(3) The applicant states that the chosen PMSS maximum water surface elevation value for the LNP site is 49.52 ft NAVD88, not the higher estimate of 49.78 ft NAVD88 obtained from the SLOSH PMSS

simulations. The PMSS maximum water surface elevation of 49.52 ft NAVD88 reported in the FSAR was obtained using an approach that the staff disagreed with previously. Also, the applicant added long-term sea-level rise and initial rise estimates after estimating the PMSS; this approach would not account for the nonlinear effects of initial water surface elevation on the PMSS.

The NRC staff requests the following additional information:

(1) The staff reviewed the applicant's approach to estimation of initial water level for a hydrodynamic storm surge model. The staff also reviewed RG 1.59, tidal data at the Cedar Key tide gauge, and NOAA's description of predicted tides. The staff determined that NOAA estimates harmonic constants at reference tide stations that are used to predict the harmonic component of tidal variations at the reference stations. Observed tide water levels also include the effects of wind wave activity and initial rise. Both of these additional effects manifest as random variations added to the harmonic component of the tidal variations. Because these random variations are independent of the harmonic forcings (mainly gravitational forces of the sun and the moon) and therefore can occur at any time, there is no assurance that "high" random variations of tides would be in phase with the highs of the predicted tides. Therefore, estimating the 10 percent exceedance tide from raw tide water level observations can result in underestimation of the initial water level (represented by 10 percent exceedance of predicted tides plus initial rise). RG 1.59 does not describe how initial rise reported for various locations in Appendix C of RG 1.59 was estimated.

The staff needs the following information to complete its review of the PMSS at the LNP site:

a. A detailed description of the applicant's approach used to estimate the initial water level for use in the SLOSH model runs, an analysis of how this approach is consistent with the recommendations of RG 1.59, a statement of the difference in the numerical values of the initial water level obtained by the applicant's approach and that recommended by RG 1.59, and a detailed justification of why the difference between the two numerical values would result in an insignificant difference in the PMSS maximum water surface elevation at the LNP site, or

b. An updated PMSS maximum water surface elevation at the LNP site that is a combination of (i) maximum stillwater elevation from a SLOSH simulation carried out with an initial water surface elevation estimated following the guidelines of RG 1.59 and using more recent tide data and (ii) wind wave effects using the CEM approach (see (2) below).

(2) Provide an update to FSAR text that clearly describes how the CEM approach was used to estimate wind wave activity coincident with PMSS maximum water surface elevation at the LNP site.

(3) Provide updates to FSAR that describe appropriately selected PMSS characteristics at the LNP site. Provide a discussion of available margins between the DCD Maximum Flood Level site parameter (the design grade elevation or the DCD plant elevation of 100 ft) and the highest PMSS water surface elevation accounting for coincident wind-wave activity.

The applicant responded to the staff's RAI 02.04.05-11 in a letter dated June 21, 2011 (ML11175A300). To address part (1) of the staff's request, the applicant performed an updated PMSS maximum water surface elevation at the LNP site by estimating an initial water surface elevation for the SLOSH model following the guidance in RG 1.59 (NRC 1977a) and using more recent tide data. Because the applicant has followed guidance in RG 1.59 (NRC 1977a) and used more recently available tide data to specify an initial water surface elevation for the SLOSH model simulation, the staff concluded that the applicant's approach for estimating the PMSS maximum water surface elevation is appropriate. The applicant found that the two methods yielded values that were close, with the larger being 0.82 m (2.68 ft) NAVD88. The applicant used this larger value for subsequent analysis. The applicant determined an initial water level for use with the SLOSH model. The applicant's initial water level was 1.18 m (3.87 ft) NAVD88, which is based on an initial rise of 0.18 m (0.60 ft), a long-term sea level rise of 0.18 m (0.59 ft), and the 10 percent exceedance tide of 0.82 m (2.68 ft) NAVD88. The applicant stated that its initial water level was slightly larger than the one used previously (1.16 m [3.82 ft] NAVD88). The applicant applied the SLOSH model with the revised initial water elevation and found it has an insignificant effect on the SLOSH model predictions for the case producing the maximum surge elevation previously reported. The applicant reported a maximum surge elevation of 14.53 m (47.7 ft) NAVD88. The staff concluded that the applicant has adequately addressed the PMSS maximum stillwater surface elevation. The staff's evaluation of issues related to wave action is described below.

2.4.5.4.3 Wave Action

Information Submitted by the Applicant

The applicant estimated that the limiting wave period would be approximately 10 seconds assuming a deep water depth of 10 m (32.8 ft). The applicant also assumed the ground surface elevations would vary between 1.5 and 4.6 m (5 and 15 ft) and the storm surge elevations would vary from 6.1 to 10.7 m (20 to 35 ft). The applicant carried out 1,000 wave setup estimations from randomly selected combinations of ground surface and storm surge elevations. The applicant selected the maximum of these 1,000 simulated wave setups, 2.3 m (7.65 ft), as the wave setup value for the LNP site. The applicant stated that the surge boundary remains to the west of U.S. Highway 19, which is approximately 6.4 km (4 mi) from the LNP site. The applicant concluded, therefore, that the temporary increase in water level was highly unlikely to reach the LNP site.

The applicant reported the total water depth as the sum of Stillwater depth and wave setup. The applicant performed 1,000 simulations for the total water depth by combining the random selection of storm surge parameters and the wave setup parameters. The maximum of the 1,000 applicant-estimated total water depths was 14.93 m (48.98 ft) NGVD29 or 14.62 m (47.98 ft) NAVD88.

NRC Staff's Technical Evaluation

The staff requested additional information regarding the methodology used in the analysis of coincident wind-generated wave action and runup in **RAI 02.04.05-07**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and10 CFR Part 100, an estimate of wind-induced wave runup under PMH winds is needed. Criteria and methods of the USACE, as generally summarized in the USACE Coastal Engineering Manual, are used as a standard to evaluate the applicant's estimate of coincident wind-generated wave action and runup. These criteria are also used to evaluate flooding, including the static and dynamic effects of broken, breaking, and nonbreaking waves. Please add a reference in the FSAR for the methodology used to estimate wave action in Lake Rousseau, or explain why such a reference is not needed.

The applicant responded to the staff's RAI 02.04.05-07 in a letter dated July 20, 2009 (ML092030128). The applicant stated that due to the narrow and irregular shape of Lake Rousseau, the fetch length in the lake would be too short to generate a wave that would affect the LNP site. As stated above, the staff determined the meteorologically or seismically generated waves in Lake Rousseau would be limited by fetch and by water depth and would not reach the LNP site.

To ensure that the applicant has considered wave runup during PMH storm surge flooding, the staff issued **RAI 02.04.05-08**, which states:

> To meet the requirements of GDC 2, 10 CFR 52.17, and 10 CFR Part 100, an estimate of wind-induced wave runup under PMH winds is needed. The applicant added the estimated wave setup to the estimated stillwater PMH storm surge to obtain total water depth at the LNP site during the PMH conditions. Please provide an estimate of wave runup during the PMH storm surge at the LNP site.

The applicant responded to the staff's RAI 02.04.05-08 in a letter dated July 20, 2009 (ML092030128). The applicant provided an estimate of wave runup under PMH conditions using the procedures described by the USACE CEM (Scheffner 2008). The applicant estimated that the maximum wave runup would be 0.26 m (0.85 ft). The applicant stated that the FSAR would be updated to include the runup analysis.

The staff reviewed the applicant's response to RAI 02.04.05-08 and its calculations to determine that the applicant has used the USACE CEM (Scheffner 2008) guidance for estimation of wave runup during PMH conditions. The staff determined that the USACE CEM (Scheffner 2008) guidelines are widely used in engineering practice and are suitable for use in estimation of site characteristics for an FSAR. The staff finds that the applicant appropriately considered wave runup during PMH conditions at the LNP site.

To determine whether the applicant has followed an approach that is consistent with the regulatory guidance in National Weather Service Report 23, the staff issued **RAI 02.04.05-11**, which states:

> In RAI 2.4.5-10, the staff requested the applicant to provide supplemental information; the staff stated that the applicant must (1) use a set of plausible probable maximum hurricane (PMH) scenarios consistent with the National Oceanic and Atmospheric (NOAA) National Weather Service (NWS) Report 23 (NWS 23) as input to a currently accepted storm surge model (such as NWS Sea, Lake, and Overland Surges from Hurricanes [SLOSH]), (2) use initial open-water conditions that are consistent with current understanding of long-term sea-level rise and are valid for the life of the proposed plants, (3) provide estimates of coincident wind-wave runup, (4) provide maps of highest probable maximum storm surge (PMSS) water surface elevation at and near the LNP sites, and (5) provide updates to FSAR Section 2.4.5, including descriptions of data, methods, model setup, PMH scenarios and how they are consistent with NWS 23, treatment of uncertainty in the analysis, and available margins.
>
> The applicant responded to RAI 2.4.5-10 on January 27, 2011. The staff's review of the applicant's response to RAI 2.4.5-10 has raised the following issues:
>
> (4) Regulatory Guide (RG) 1.59 recommends that the following components of PMSS be estimated: (a) probable maximum surge (wind and pressure setups), (b) 10 percent exceedance tide, and (c) initial rise (forerunner or sea-level anomaly). The wind wave runup also needs to be added to obtain the PMSS. The applicant did not use an initial rise in its SLOSH simulations. RG 1.59 recommends an initial rise of 0.6 ft for Crystal River, FL. Because the value of initial water surface can have nonlinear effects on SLOSH predictions, 10 percent exceedance tide, initial rise, and long-term sea level rise should be combined to specify the initial water surface in SLOSH for simulation of the PMH scenarios.
>
> In a subsequent teleconference, the applicant stated its interpretation of RG 1.59 recommendations. The applicant stated that RG 1.59 recommends use of initial rise as an additional component of the initial water level if the 10 percent exceedance tide is estimated from predicted tides. The applicant stated that use of initial rise is not necessary because its approach used observations of tidal water levels that already contain the effects of initial rise.

(5) The applicant has not used the US Army Corps of Engineers Coastal Engineering Manual (CEM) for estimation of coincident wind wave activity. The CEM approach is recommended in SRP 2.4.5 as the currently accepted practice. The applicant did not provide justification why it used another approach. In a subsequent teleconference, the applicant stated that they did in fact use the CEM approach to estimate wind wave activity although this fact was not clearly stated in the response to RAI 2.4.5-10.

(6) The applicant states that the chosen PMSS maximum water surface elevation value for the LNP site is 49.52 ft NAVD88, not the higher estimate of 49.78 ft NAVD88 obtained from the SLOSH PMSS simulations. The PMSS maximum water surface elevation of 49.52 ft NAVD88 reported in the FSAR was obtained using an approach that the staff disagreed with previously. Also, the applicant added long-term sea-level rise and initial rise estimates after estimating the PMSS; this approach would not account for the nonlinear effects of initial water surface elevation on the PMSS.

The NRC staff requests the following additional information:

(4) The staff reviewed the applicant's approach to estimation of initial water level for a hydrodynamic storm surge model. The staff also reviewed RG 1.59, tidal data at the Cedar Key tide gauge, and NOAA's description of predicted tides. The staff determined that NOAA estimates harmonic constants at reference tide stations that are used to predict the harmonic component of tidal variations at the reference stations. Observed tide water levels also include the effects of wind wave activity and initial rise. Both of these additional effects manifest as random variations added to the harmonic component of the tidal variations. Because these random variations are independent of the harmonic forcings (mainly gravitational forces of the sun and the moon) and therefore can occur at any time, there is no assurance that "high" random variations of tides would be in phase with the highs of the predicted tides. Therefore, estimating the 10 percent exceedance tide from raw tide water level observations can result in underestimation of the initial water level (represented by 10 percent exceedance of predicted tides plus initial rise). RG 1.59 does not describe how initial rise reported for various locations in Appendix C of RG 1.59 was estimated.

The staff needs the following information to complete its review of the PMSS at the LNP site:

a. A detailed description of the applicant's approach used to estimate the initial water level for use in the SLOSH model runs, an analysis of how this approach is consistent with the recommendations of RG 1.59, a statement of the difference in the numerical values of the initial water level obtained by the applicant's approach and that recommended by RG 1.59, and a detailed justification of why the difference between the two numerical values would result in an insignificant difference in the PMSS maximum water surface elevation at the LNP site, or

b. An updated PMSS maximum water surface elevation at the LNP site that is a combination of (i) maximum stillwater elevation from a SLOSH simulation carried out with an initial water surface elevation estimated following the guidelines of RG 1.59 and using more recent tide data and (ii) wind wave effects using the CEM approach (see (2) below).

(5) Provide an update to FSAR text that clearly describes how the CEM approach was used to estimate wind wave activity coincident with PMSS maximum water surface elevation at the LNP site.

(6) Provide updates to FSAR that describe appropriately selected PMSS characteristics at the LNP site. Provide a discussion of available margins between the DCD Maximum Flood Level site parameter (the design grade elevation or the DCD plant elevation of 100 ft) and the highest PMSS water surface elevation accounting for coincident wind-wave activity.

The applicant responded to the staff's RAI 02.04.05-11 in a letter dated June 21, 2011 (ML11175A300). The applicant's response to part (1) of the staff's request and the staff's review of the applicant's response to part (1) are described above in Section 2.4.5.4.2 of this SER.

To address part (2) of the staff's request, the applicant used the Automated Coastal Engineering Systems (ACES) software to compute wave action at the LNP site. The applicant states that the software is designed to use the methods outlined in the USACE CEM (Scheffner 2008). The applicant states that due to the shallowness of water at the LNP embankment and the high wind conditions the waves at the LNP site will break. The applicant then uses breaking-wave calculations to estimate wave runup. The applicant estimated a wind-wave setup of 0.18 m (0.6 ft). Using the SLOSH-predicted PMSS maximum water elevation of 14.5 m (47.7 ft) NAVD88 combined with the wind setup of 0.18 m (0.6 ft), the applicant estimated that the water depth at the toe of an affected structure located at a grade elevation of 14.3 m (47.0 ft) NAVD88 would be 0.4 m (1.3 ft). The applicant used USACE CEM (Scheffner 2008) guidance the water depth to compute a wave period of 1.96 seconds and, along with the wave-breaking assumption, estimated a maximum wave height of 0.3 m (1.0 ft). The applicant found that for these conditions, ACES yielded a 0.45–m (1.48–ft) maximum wave runup. The applicant stated that updates to the FSAR based on the approach outlined in the RAI response will be made. The staff concluded that the applicant has adequately addressed the issue related to the estimation of PMH wind-wave action at the site. The staff is tracking future FSAR updates as **Confirmatory Item 2.4.5-1**.

<u>Resolution of Confirmatory Item 2.4.5-1</u>

Confirmatory Item 2.4.5-1 is an applicant commitment to update Section 2.4.5 of its FSAR. The staff verified that LNP COL FSAR Section 2.4.5 was appropriately updated. As a result, Confirmatory Item 2.4.5-1 is now closed.

The applicant responded to part (3) of this request with a discussion of the available margin between the DCD maximum flood level and the maximum estimated PMH surge level. The applicant stated that the maximum flood level as the sum of the maximum PMH surge level (14.54 m [47.7 ft] NAVD88), the initial rise (0.18 m [0.6 ft]), and the maximum wave runup (0.45 m [1.48 ft]) or 15.17 m (49.78 ft) NAVD88. The applicant stated that the LNP DCD plant elevation is 15.54 m (51 ft) NAVD88, leaving a margin of 0.37 m (1.22 ft).

The staff reviewed the methods used by the applicant in estimation of the maximum PMSS water surface elevation and concluded that it is acceptable because the applicant has used current guidance supplemented with more recently available data and used conservative assumptions. Therefore, the staff has determined that the applicant has adequately addressed the effects of the PMH on the water surface elevation at the LNP site.

2.4.5.4.4 Resonance

<u>Information Submitted by the Applicant</u>

The applicant stated that adverse effects from resonance in Lake Rousseau and the Gulf of Mexico on safety-related SSCs at the LNP site appear to be unlikely because the resonance will be quickly dissipated.

<u>NRC Staff's Technical Evaluation</u>

The staff reviewed the applicant's response to RAI 02.04.05-06 to evaluate the effects of resonance in Lake Rousseau and any induced flood wave that may travel from the lake towards the LNP site. As stated above, the staff determined the meteorologically or seismically generated waves set up in Lake Rousseau would be limited by fetch and by water depth and would not reach the LNP site. The staff considers RAI 02.04.05-06 to be resolved.

2.4.5.4.5 Protective Structures

<u>Information Submitted by the Applicant</u>

The applicant stated that all safety-related SSCs are protected from adverse effects of water up to an elevation of 51 ft NAVD88, which is higher than the design basis flood at the LNP site.

<u>NRC Staff's Technical Evaluation</u>

The staff evaluated the highest floodwater elevations during PMH conditions resulting from storm surge, wave setup, and wave runup to determine if all safety-related SSCs are adequately protected after the review of the applicant's responses to RAIs 02.04.05-09, 02.04.05-10, and 02.04.05-11. The staff has accepted the applicant's conclusion that the design-basis flood elevation at the LNP site is caused by a PMH and results in a combined effects maximum water surface elevation of 15.17 m (49.78 ft) NAVD88, which is lower than the LNP site grade elevation of 15.24 m (50 ft) NAVD88 and the corresponding DCD plant elevation of 15.54 m (51 ft) NAVD88 with an available margin of 0.37 m (1.22 ft).

The staff has completed its review of the maximum water surface elevations near the LNP site after the applicant's PMH analysis was completed as documented by the responses to RAIs 02.04.05-09, 02.04.05-10, and 02.04.05-11. Therefore, the staff considers these RAIs to be resolved.

2.4.5.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.5.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to probable maximum surge and seiche flooding, and that there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.5, of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses part of COL information item 2.4-2.

2.4.6 Probable Maximum Tsunami Hazards

2.4.6.1 *Introduction*

The probable maximum tsunami hazards are addressed to ensure that any potential tsunami hazards to the SSCs important to safety are considered in plant design. The specific areas of review are as follows: (1) historical tsunami data, including paleotsunami mappings and interpretations, regional records and eyewitness reports, and more recently available tide gauge and real-time bottom pressure gauge data, (2) probable maximum tsunami (PMT) that may pose hazards to the site, (3) tsunami wave propagation models and model parameters used to simulate the tsunami wave propagation from the source towards the site, (4) extent and duration of wave runup during the inundation phase of the PMT event, (5) static and dynamic force metrics, including the inundation and drawdown depths, current speed, acceleration, inertial component, and momentum flux that quantify the forces on any safety-related SSCs that may be exposed to the tsunami waves, (6) debris and water-borne projectiles that accompany tsunami currents and may impact safety-related SSCs, (7) effects of sediment erosion and deposition caused by tsunami waves that may result in blockage or loss of function of safety-related SSCs, (8) potential effects of seismic and non-seismic information on the postulated design bases and how they relate to tsunami in the vicinity of the site and the site region, and (9) any additional information requirements prescribed within the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.6.2 *Summary of Application*

This section of the COL FSAR addresses the site-specific information about potential dam failures. The applicant addressed the information as follows:

AP1000 COL Information Item

- LNP COL 2.4-6

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 19 of the AP1000 DCD.

> Combined License applicants referencing the AP1000 design will address the following site-specific information on historical flooding and potential flooding factors, including the effects of local intense precipitation.
>
> - Probable Maximum Flood on Streams and Rivers – Site-specific information that will be used to determine design basis flooding at the site. This information will include the probable maximum flood on streams and rivers.
> - Dam Failures – Site-specific information on potential dam failures.
> - Probable Maximum Surge and Seiche Flooding – Site-specific information on probable maximum surge and seiche flooding.
> - Probable Maximum Tsunami Loading – Site-specific information on probable maximum tsunami loading.
> - Flood Protection Requirements – Site-specific information on flood protection requirements or verification that flood protection is not required to meet the site parameter of flood level.

2.4.6.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification of tsunami floods, tsunami flood design considerations and the associated acceptance criteria, are described in Section 2.4.6 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying the effects of tsunami flooding are:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d) sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Appropriate sections of the following RGs are used by the staff for the identified acceptance criteria:

- RG 1.59, "Design Basis Floods for Nuclear Power Plants" (NRC 1977a), as supplemented by best current practices; and

- RG 1.102, "Flood Protection for Nuclear Power Plants" (NRC 1976b).

2.4.6.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.6 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the probable maximum tsunami hazards. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

2.4.6.4.1 Probable Maximum Tsunami

<u>*Information Submitted by the Applicant*</u>

Because the applicant did not include a summary of the PMT assessment in Section 2.4.6.1 of the FSAR, information from other sections of the FSAR was used to determine which sources were considered and what the applicant determined were the water levels associated with each source. Three tsunami source regions were considered by the applicant to determine the PMT: (1) far-field sources outside the Gulf of Mexico and Caribbean region, (2) seismogenic sources along the Caribbean plate boundary, and (3) earthquake and landslide tsunami sources in the Gulf of Mexico. For the far-field sources, the applicant appears to consider that the maximum wave height would be from an event similar to the 1755 Lisbon seismogenic tsunami (<1 m wave heights in the Gulf of Mexico). For Caribbean sources, the worst-case scenario is determined by the applicant to be a seismogenic tsunami offshore Venezuela (in the Caribbean Sea), with a maximum wave height of 0.65 m offshore of the site (FSAR pg. 2.4-58). For Gulf of Mexico tsunami sources, the applicant considered the East Breaks slump in the northwest Gulf of Mexico as the worst-case scenario, with a maximum wave height of 1.68 m offshore of the site (FSAR pg. 2.4-53). The applicant stated that the controlling source of the PMT appears to be the East Breaks landslide.

To obtain clarification on the most reasonably severe geo-seismic activity possible and corresponding tsunami analysis, the staff issued **RAI 02.04.06-01**, asking the applicant for a summary of the PMT assessment for the Levy County site, including the controlling source for the PMT and corresponding tsunami water level determination. The applicant responded to the staff's RAI 02.04.06-01 in a letter dated July 22, 2009 (ML092080077). The applicant refers to the responses of RAI 02.04.06-08 and 02.04.06-10, suggesting that the Mississippi Canyon slide is the controlling source for the PMT. The PMT runup indicated in the response to RAI 02.04.06-01 does not agree with either the uncorrected or corrected PMT runup values indicated in the applicant's responses to RAI 02.40.6-06 (Tables 1 and 2), RAI 02.04.06-08 (Table 3), and RAI 02.04.06-10 (Table 1).

The applicant responded to the staff's RAI 02.04.06-11 in a letter dated March 25, 2010. The applicant states that the PMT runup and run-in values for a Mississippi Canyon-like slide moving down slope at a velocity of 50 m/s (164 ft/s) were incorrectly presented as 23.5 m (77.1 ft) NAVD88 and 2.19 km (1.36 mi), respectively. The correct PMT runup and run-in values are 22.5 m (73.8 ft) NAVD88 and 2.07 km (1.29 mi), respectively, as presented in the response to RAI 2.4.6-10 (Table 1). The associated LNP COL in FSAR Subsection 2.4.6, Rev. 1 was revised to incorporate clarification of the PMT analysis and text presented in LNP calculation package LNG-0000-X7C-043, Revision 0. The correct PMT runup and run-in values presented above was also included in this revision. Therefore, the staff considers RAIs 02.04.06-01 and 02-04-06-11 to be resolved.

To obtain information on the generation of tsunami-like waves from hill-slope failures and the stability of the coastal area, the staff issued **RAI 02.04.06-02**, asking the applicant to provide a discussion of the generation of tsunami-like waves from hill-slope failures and the stability of the coastal area in the updated FSAR with reference to the findings in Section 2.5 of the FSAR. The applicant responded to the staff's RAI 02.04.06-02 in letters dated July 22, 2009 (ML092080077) and August 09, 2010 (ML102290085). The applicant stated that no permanent slopes or hill slopes are present near the site or within the coastal areas near the site. Therefore, the staff considers RAI 02.04.06-02 to be resolved.

NRC Staff's Technical Evaluation

The NRC Staff conducted an independent confirmatory analysis to determine the PMT at the Levy County site that is described in detail in the sections that follow. In summary, numerical hydrodynamic modeling of three different types of tsunami sources have been performed to determine their impact on the Levy County site. The three source types are (1) distant earthquake sources; (2) a regional earthquake source in the Gulf of Mexico; and (3) regional submarine landslide sources in the Gulf of Mexico. Most of the analysis is focused on source type (3) for determination of the PMT. For all conditions, the most conservative source parameters were employed, even when arguably unphysical, to provide an absolute upper limit on the possible tsunami effects at the Levy County site.

The Staff found that the applicant did not use any of the standard methods of tsunami propagation and inundation modeling. In RAI 2.4.6-08, the staff requested additional information regarding the applicant's analysis procedure used to calculate tsunami wave height and period at the site, including the theoretical bases of the models, their verification and the conservatism of all input parameters. In a letter dated July 22, 2009, the applicant describes a procedure in which an estimated source amplitude is multiplied by three factors: (1)propagation loss, (2) shoaling correction, and (3) "beaching" amplification. Each of the multiplicative factors is determined from analytic expressions—variations in water depth along the propagation path between the source and the site were not explicitly accounted for. The results of their analysis indicate that the PMT is from a Mississippi Canyon landslide source, with a maximum water level of 21.4 m (Response to RAI 02.04.06-8). Including sea-level rise, sea-level anomaly, and high tide, their PMT maximum water level is 22.5 m (NAVD88) (Response to RAI 02.04.06-10), substantially above the plant grade elevation of 15.5 m (NAVD88).

Using conservative source parameters and neglecting the radial spreading of wave energy, the staff's 1HD simulations indicate that the Mississippi Canyon source clearly has the greatest potential to bring at large wave to the Levy site, with 1HD water elevations near the site in excess of +30 m. The staff's 2HD simulations of this source and the WORST CASE Florida Slope landslide source that include radial spreading predict a maximum wave elevation of 7 m offshore of the site (30 m water depth). However, the Mississippi Canyon wave is longer in period and has a longer train of large waves, and thus is designated as the PMT for the Levy site. The staff's highly refined nearshore simulations show that this source results in a maximum water level of +3 m. Because of nonlinear effects during wave propagation, one cannot simply add an antecedent sea level that includes 10 percent exceedance high tide, sea level anomaly, and sea-level rise to this maximum water to the +3m maximum water level. A separate simulation that includes the nonlinear propagation effects and a +1.2 m (NAVD88) antecedent sea level results in a maximum water level of +6.1 m. Thus, the results from the staff's independent analysis indicate that the PMT does not reach the Levy site plant grade elevation. Therefore, the staff considers RAI 2.4.6-8 to be resolved.

2.4.6.4.2 Historical Tsunami Record

Information Submitted by the Applicant

The applicant reviews tsunami catalogs for the Caribbean and the Gulf of Mexico regions and determines that there were three events that affected the Gulf coast: two seismogenic tsunamis and one seismic seiche. The sources of information primarily include the NOAA/NGDC Historical Tsunami Database (internet) and the published report of Lander et al. (2002).

The first seismogenic tsunami was caused by the 1918 Mona Passage earthquake, located northwest of Puerto Rico. Maximum runup from the tsunami was reported to be 6 m local to the source. In the Gulf of Mexico, the tsunami was recorded at the Galveston tide gauge station, but the maximum amplitude of the wave was not indicated by the applicant.

The second seismogenic tsunami was caused by an earthquake near Vieques Island in 1922. In the Gulf of Mexico, a maximum amplitude of 0.6 m was recorded at the Galveston tide gauge station, with a dominant period of 45-minutes.

A seiche was observed in the Gulf of Mexico in 1964 that was set up by seismic waves emanating from the 1964 Gulf of Alaska earthquake. The applicant did not indicate the maximum amplitude of the seiche in the Gulf of Mexico.

To obtain clarification with respect to the historical tsunami record, the staff issued RAIs 02.04.06-03, 02.04.06-04 and 02.04.06-05. In RAI 02.04.06-03, the staff asked the applicant to provide clarification in the updated FSAR of the meaning of the descriptor “impact” as used on pg. 2.4-45 of the FSAR: “…historically no Caribbean tsunami has *impacted* the United States Gulf Coast.” The applicant responded to the staff’s RAI 02.04.06-03 in letters dated July 22, 2009 (ML0920800771), and August 09, 2010 (ML1022900851). The applicant explains in their response that the descriptor “impact” means “no tsunamis are known to have originated in the Caribbean Sea and generated a runup exceeding 1.0 m at any location along the United States Gulf Coast.” Therefore, the staff considers RAI 02.04.06-03 to be resolved.

The staff issued RAI 02.04.06-04 to provide clarification in the updated FSAR whether any of the Maximum Water Height measurements listed in FSAR Table 2.4.6-202 are located in the Gulf of Mexico. The applicant responded to the staff’s RAI 02.04.06-04 in a letter dated July 22, 2009 (ML0920800771). The applicant indicates that none of the locations of Maximum Water Height measurements are located in the Gulf of Mexico. It should be noted that the Maximum Water Height measurements are typically located near the source—not necessarily in the Caribbean as the applicant indicates in their response to RAI 2.4.6-04. Therefore, the staff considers RAI 02.04.06-04 to be resolved.

The staff issued RAI 02.04.06-05, asking the applicant to provide clarification in the updated FSAR whether there is any geologic evidence of tsunami deposits at the Levy County site or at nearby regions. Additionally, indicate whether there are geologically conducive locations for the deposition and preservation of tsunami deposits in the vicinity of the Levy County site. If such paleo-tsunami evidence exists, indicate how they are distinguished from storm wash-over deposits. The applicant responded to the staff’s RAI 02.04.06-05 in a letter dated July 22, 2009 (ML0920800771). The applicant indicates that site-specific borings lead them to conclude that there is no geologic evidence of paleo-tsunami or tsunami-like deposits in the vicinity of the Levy County site. However, the applicant needs to provide additional details of the sedimentological analysis used to arrive at this conclusion, including the thickness of sand layers that the methods used were capable of detecting, and cross reference to applicable parts of FSAR Section 2.5. The applicant responded to the staff’s RAI 02.04.06-12 in a letter dated March 25, 2010 (ML100910299), with additional details of the sedimentological analysis. Based on the applicant’s detailed response, the staff considers RAIs 02.04.06-05 and 02.04.06-12 to be resolved.

NRC Staff’s Technical Evaluation

The Staff reviewed the applicant’s primary references of historical observations and measurements of tsunami and seismic seiche waves occurring along the Gulf Coast and finds the applicant’s assessment of the historical tsunami record to be acceptable.

The closest locations of interpreted paleotsunami deposits to the Levy County site are in southern Alabama, as shown in FSAR Figure 2.4.6.4.2-1. The deposits are thought to be part of a regional tsunami event in the Gulf of Mexico at or near the time of the Cretaceous-Tertiary (K-T) boundary.

The common interpretation of this deposit is that it was emplaced by a tsunami generated from Chicxulub asteroid impact, owing to its date and the existence of impact ejecta at the Brazos site and elsewhere. However, the tsunami deposit was discovered by Bourgeois et al. (1988) prior to the discovery of the Chicxulub impact crater (Hildebrand and others, 1991). An important alternate hypothesis related to possible tsunamigenic sources in the Gulf of Mexico is provided by Bourgeois et al. (1988):

> "If the tsunami were produced by a major submarine landslide, it should not occur precisely at the K-T boundary unless the landslide were caused by an earthquake related to boundary events, which is a possibility" (pg. 569)

Bourgeois et al. (1988) suggested that a tsunami wave 50-100 m high was necessary to explain this deposit. The published wave heights and flow speeds of the Brazos tsunami deposit are reasonable, representing order-of-magnitude estimates. It is not conceivable that the wave that created these deposits was generated by any landslide source that would be of relevance to the present-day PMT determination. As the staff demonstrates in independent analysis, any landslide wave generated at the present-day continental shelf break would not be able to maintain a large wave height across such a long propagation distance over very shallow water. The depth-limiting dissipation effect, in which large amplitude waves are dissipated much faster than small amplitude waves during long propagation over shallow depth, would necessarily reduce any landslide generated wave located at the shelf break to a minimal event at the shoreline. It is still possible that this deposit was generated by a paleo-landslide source, but this landslide event would have been local to the Brazos site. It is considerably more likely that a wave of the estimated height would be caused by a relatively nearby large impact event. Waves emanating from such a source would have the needed extreme wave heights and long periods to be able to propagate significant wave energy this far inland.

Over the last 20 years, the Brazos deposit has been extensively sampled from out crops and subsurface cores at sites near the banks of the Brazos River. Recently, studies have both corroborated and disputed whether the Brazos deposit was emplaced by a tsunami, whether it occurred exactly at the geologic boundary between the Cretaceous and Tertiary periods (i.e., at the K-T boundary), and whether the trigger was the Chicxulub impact (e.g., Smit and others, 1996; Gale, 2006; Schulte and others, 2006; Keller and others, 2007). Conflicting interpretations of the deposits at the southern Alabama locations are described in earlier studies (Mancini and others, 1989; Liu and Olsson, 1992; Savrda, 1993; Keller and Stinnesbeck, 1996). The exact age and hydrologic process that formed the regional tsunami deposit remain controversial. However, in light of these studies over the last 20 years, the lead author of original study identifying the deposit maintains that it was emplaced by a tsunami (J. Bourgeois, pers. comm., 2009).

The Staff examined primary references of historical observations and measurements of tsunami and seismic seiche waves occurring along the Gulf Coast were examined.

The applicant did not provide evidence that an adequate investigation was conducted for tsunami deposits at or near the proposed site. Additionally, the applicant does not consider the existence of a possible paleotsunami (Bourgeois and others, 1988) that occurred along the ancient Gulf Coast shoreline, including locations in southern Alabama. The common interpretation of this deposit is that it was emplaced by a tsunami generated by the Chixulub impact or by landslide or earthquake activity associated with the impact. Although arguments have been presented against this interpretation, this deposit, along with the historical record, should be considered as possible evidence of tsunami occurrence along the Gulf Coast. However, the staff finds that the flow speeds and wave heights inferred from the deposit are not relevant to determination of the present-day PMT.

2.4.6.4.3 Source Generator Characteristics

Information Submitted by the Applicant

The applicant identifies possible tsunami sources from three general regions: (1) far-field sources outside of the Gulf of Mexico and Caribbean Sea, (2) the Caribbean plate boundary, and (3) inside the Gulf of Mexico.

Far-field source scenarios initially considered include the 1964 Gulf of Alaska seismic seiche, the 1755 Lisbon seismogenic tsunami, and far-field landslide sources in the Atlantic Ocean. The applicant appears to consider only the 1755 Lisbon seismogenic in determining water levels from a far-field source.

Caribbean sources include earthquakes along the boundary of the Caribbean plate. Specific earthquake and tectonic segments considered by the applicant include the North Panama Deformation Belt, the northern South America convergence zone, the northern Caribbean subduction zone, and the Cayman transform fault system.

Gulf of Mexico tsunami sources considered include intra-plate earthquakes and landslides. For intra-plate earthquakes, the applicant indicates the historical occurrence of the Mw=5.8 September 10, 2006 Gulf of Mexico earthquake, but does not include a seismogenic source in this region of the Gulf of Mexico in their tsunami analysis. The applicant does include the results from a scenario by Knight (2006) offshore Veracruz, Mexico, that the applicant links to present-day seismic activity. For landslides in the Gulf of Mexico, the applicant primarily considers the East Breaks landslide offshore Texas, but not other possible landslide sources in the Gulf of Mexico. All of the aforementioned information was obtained by the applicant from published journal articles and web sites.

In order to obtain a more comprehensive picture of tsunami source generators, the staff issued RAIs 02.04.06-06 and 02.04.06-07. In RAI 02.04.06-06, the staff asked the applicant to provide a discussion in the updated FSAR of submarine landslides in the Gulf of Mexico, other than East Breaks, as potential tsunami generators, including the Mississippi Canyon landslide, and landslides along the Florida Escarpment and along the slope above the Florida Escarpment. In addition, clarify text in the FSAR indicating whether the East Breaks landslide is considered as the PMT source, in relation to discussion of the north Venezuela seismogenic tsunami as having "the most severe impacts for the Gulf Coast" (pg. 2.4-58).

The applicant responded to the staff's RAI 02.04.06-06 in a letter dated July 22, 2009 (ML0920800771). In their response to RAI 02.04.6-06, the applicant is inconsistent in their characterization of the Mississippi Canyon and Florida Escarpment tsunami sources. On page 9-10 of their response, the applicant appears to discount the tsunami potential based on the date of the last landslides in those regions. In the rest of their response, they indicate that these sources are used for PMT determination (and, in fact, the Mississippi Canyon slide is the applicant's controlling PMT source). The applicant needs to clarify whether the Mississippi Canyon and Florida Escarpment are considered to be significant potential sources for PMT determination. In addition, the applicant indicates identical source parameters for "Florida Escarpment" and "Slope above the Florida Escarpment" in Table 1 of their response to RAI 02.04.6-06. However, the water depth in these two regions is different. The applicant needs to explain this apparent discrepancy, or justify why the entries in Table 1 are correct. The applicant responded to the staff's RAI 02.04.06-13 in a letter dated March 25, 2010 (ML1009102991), with additional details and a revised Table 1. Based on the applicant's detailed response and FSAR revision, the staff considers RAIs 02.04.06-06 and 02.04.06-13 to be resolved.

The staff issued RAI 02.04.06-07, asking the applicant to provide clarification in the updated FSAR regarding seismologic characterization of the region offshore Veracruz, Mexico, relative to the generation of tsunamis. The applicant responded to the staff's RAI 02.04.07 in a letter dated July 22, 2009 (ML0920800771). The applicant's explanation provides additional details of the source parameters considered, although the staff is not aware of 15-20 earthquakes > M7 near Veracruz Mexico. The applicant needs to clarify the location of "15-20 earthquakes of magnitude 7 or greater…near Veracruz" indicated in the applicant's response to RAI 02.04.06-07, in terms of tsunami potential for the Gulf of Mexico versus the Pacific Ocean. The applicant should also provide the information source for this statement. The staff issued RAI 02.04.06-14 to obtain additional information related to the "15-20 earthquakes of magnitude 7 or greater…near Veracruz" described in the applicant's response to RAI 02.04.06-07. The applicant responded to the staff's RAI 02.04.06-14 in a letter dated March 25, 2010 (ML1009102991), with additional geo-seismic descriptions of controlling distant tsunami generators, including location, source dimensions, fault orientation, and maximum displacement. Based on the applicant's detailed response, which conforms to the guidance in section C.I.2.4.6.3 of RG 1.206, the staff considers RAIs 02.04.06-07 and 02.04.06-14 to be resolved.

NRC Staff's Technical Evaluation

In this section, tsunami sources used for the independent confirmatory analysis are described in terms of their identification, characteristic, and tsunami generation parameters. Potential tsunamigenic sources are first discussed below, including parameters associated with the maximum submarine landslides in the Gulf of Mexico. At the end of this section, we briefly discuss seismic seiches.

Potential tsunami sources that are likely to determine the PMT at the Levy County site are submarine landslides in the Gulf of Mexico. Subaerial landslides, volcanogenic sources, near-field intra-plate earthquakes and inter-plate earthquakes along Caribbean plate boundary faults are unlikely to be the causative tsunami generator for the PMT at the Levy County site as discussed below.

With regard to subaerial landslides, there are no major coastal cliffs near the site that would produce tsunami-like waves that exceed the amplitude of those generated by other sources.

Volcanogenic Sources

According to the Global Volcanism Program of the Smithsonian Institution (http://www.volcano.si.edu/), there are three general regions of volcanic activity that have the potential to generate localized wave activity in the Gulf of Mexico and Caribbean Sea: (1) two Mexican volcanoes near the Gulf of Mexico coastline; (2) two volcanoes in the western Caribbean; and (3) volcanic activity along the Lesser Antilles island arc. Two Mexican volcanoes, (Cerro el Abra/Los Atlixos and San Martin) associated with the eastern Trans-Mexican Volcanic Belt, are located near the Gulf of Mexico coastline. Basaltic flows associated with Los Atlixcos have reached as far as the coast. Also in the eastern Caribbean, Volcán Azul on the coast of Nicaragua is composed of three small cinder cones, but these are unlikely to generate significant failures. There are many active volcanoes along the Lesser Antilles island arc, some of which have historically caused local tsunamis (Pelinovsky and others, 2004). However, catastrophic failures associated with volcanoes along the eastern coasts of Mexico and Central American are either too far inland or too small in size to generate significant wave activity in the Gulf of Mexico near the Levy County site. Based on existing evidence, volcanoes along the Lesser Antilles or in the eastern Atlantic Ocean are too far away and/or unfavorably situated to generate significant wave activity in the Gulf of Mexico.

Intra-Plate Earthquakes

Because there are no tectonic plate boundaries in the Gulf of Mexico region, earthquakes *local* to the Levy County site occur in an intra-plate tectonic environment, limiting the maximum magnitude these earthquakes can attain. According to the documentation for the 2008 update of the United States National Seismic Hazard Maps (Petersen and others, 2008), the maximum magnitude (M_{max}) for the Florida Gulf coast is estimated to be approximately M_{max}=7.5. See Wheeler (2009) and Mueller (2010) for further details. Because the maximum slip, and consequently the maximum sea floor displacement, associated with an earthquake scales with its magnitude, the initial tsunami wave amplitude associated with an intra-plate earthquake would therefore be less than that used for local, submarine landslides under the conservative hot-start conditions as described in Section 2.4.6.4.5. Empirical evidence from global earthquakes indicates that the maximum local tsunami runup from M_w=7.5 earthquakes is approximately 6 m (Geist, 2002). This maximum is related to an earthquake along an island arc (Kuril Islands) without a broad continental shelf.

Inter-Plate Earthquakes

In the far-field, offshore tsunami amplitudes from Carribbean inter-plate earthquakes are estimated in Chapter 8 of ten Brink and others (2008), using the linear-long wave equations. The description of major plate boundary faults and specific source parameters are described in that study. The tsunami propagation model presented in ten Brink and others (2008) has been refined during our confirmatory analysis for two of the principal sources (the northern South America Convergent Zone and the northern Caribbean Subduction Zone) using the COMCOT tsunami model discussed in Sections 2.4.6.4.4 and 2.4.6.4.5. Tsunami amplitudes at the Florida Gulf coast from these seismogenic sources are generally small (i.e., < 1 m) compared to tsunami amplitudes determined for submarine landslides in establishing the PMT. Tsunami amplitudes from earthquakes along the Azores-Gibraltar oceanic convergence boundary are also likely to be small (i.e., < 1 m) in the Gulf of Mexico (Mader, 2001; Barkan and others, 2009). For the remainder of this section, we focus on submarine landslide sources as the principal generator for the PMT at the Levy County site.

Submarine Landslides in the Gulf of Mexico

Submarine landslides in the Gulf of Mexico are considered a potential tsunami hazard for the Levy County site for several reasons: (1) some dated landslides in the Gulf of Mexico have post-glacial ages (Coleman and others, 1983), suggesting that triggering conditions for these landslides are still present, (2) the size and shallow initiation depth of landslides in the Gulf of Mexico, and (3) analysis of recent seismicity suggest the presence of small-scale energetic landslides in the Gulf of Mexico.

With regard to (1), the Mississippi Canyon landslide is dated 7,500-11,000 years before present (ybp) (Coleman and others, 1983; Chapter 3 in ten Brink and others, 2007) and the East Breaks landslide is dated 15,900 ± 500 ybp (Piper and Behrens, 2003). Both landslides, which are among the largest landslides in the Gulf of Mexico, occurred after the end of the last glacial maximum, during post-glacial transgression. Although landslide activity along the passive margins of North America may be decreasing with time since the last glacial period, the 1929 Grand Banks landslide is a historic example of such an event that produced a destructive tsunami (Fine and others, 2005). In addition, the Mississippi River continues to deposit large quantities of water-saturated sediments on the continental shelf and slope, making them vulnerable to over-pressurization and slope failure.

With regard to (2), several submarine landslide characteristics have been found to be significant in determining tsunami generation potential of the landslide, headwall depth including landslide volume, initial acceleration of the slide mass, and slide velocity (Ward, 2001; Harbitz and others, 2006). The volume of failed material for each of several of the landslides in the Gulf of Mexico (see below) and the shallow headwall depths (< 300 m) of the East Breaks and Mississippi Canyon landslides suggest that these landslides had the potential to generate tsunamis.

Finally, with regard to (3), seismograms of an event that occurred on February 10, 2006 (i.e., the Green Canyon event, FSAR Figure 2.4.6.4.3-2) that occurred offshore southern Louisiana (Dewey and Dellinger, 2008) suggest that energetic landslides continue to occur in the Gulf of Mexico (Nettles, 2007). Most landslides affected by salt tectonics are small in size (e.g., in comparison to the East Breaks landslide; Chapter 3 of ten Brink and others, 2007) and unlikely to be tsunamigenic. However, in terms of the failure duration, the 2006 event must have occurred rapidly enough to have generated seismic energy. While source analyses of this event cannot definitively distinguish between a fault and landslide source and evidence of significant sediment failure has not yet been found (Dellinger and Blum, 2009) this event reveals the potential for present-day slope failure.

Maximum Submarine Landslides

The NRC Staff defines four provinces in the Gulf of Mexico that are likely to be the origin of submarine landslides that control the determination of the PMT. Three additional provinces defined in Chapter 3 of ten Brink and others (2007) are not likely to be sites of major tsunamigenic landslides. The four provinces defined for PMT analysis are the Florida Escarpment and Slope region (immediately off the Levy County site), Mississippi Canyon, Northwest Gulf of Mexico, and Campeche Escarpment and Slope. The Northwest Gulf of Mexico is a mixed canyon/fan and salt province consisting of terrigenous and hemipelagic sediment, the Mississippi Canyon a canyon/fan province consisting of terrigenous and hemipelagic sediment and the Campeche and Florida margins are carbonate provinces formed from reef structures and characterized by having steep slopes. Above these escarpments a broad gentle slope comprised of carbonate sediment separates the escarpments from the shelf.

The primary landslide parameters that are used in the tsunami models include the excavation depth and slide width, which can be directly measured from sea floor mapping of the largest observed slide in the four geologic provinces. The other necessary parameter is downslope landslide length, interpreted from the runout distance. The runout distance measured from sea floor mapping is a combination of fast plug flow (low viscosity, non-turbulent), creeping plug flow (high viscosity/viscoplastic, non-turbulent) and turbidity currents (turbulent boundary layer fluid). The latter two likely have little to no tsunami-generating potential. Also, turbidity currents often involve entrainment of material during flow, such that the deposition volume may be greater than the excavation volume. Finally, hydroplaning may increase the runout of submarine landslides. The landslide lengths indicated below are intended to represent the main tsunami-generating phase. The amplitude of the initial negative wave above the excavation region is linked to the maximum excavation depth. The amplitude of the initial positive wave above the deposition region is determined from a conservation of landslide volume. The excavation volume can be well determined using GIS techniques (see below). Setting the deposition volume equal to the excavation volume, the positive amplitude is determined for a given landslide length. For a fixed volume, increasing the landslide length decreases the initial positive amplitude of the landslide tsunami.

Landslide volume calculations are based on measuring the volume of material excavated from the landslide source area using a technique similar to that applied by ten Brink and others (2006) and Chaytor and others (2009). Briefly stated, the approach involves using multibeam bathymetry to outline the extent of the excavation area, interpolating a smooth surface through the polygons that define the edges of the slide to provide an estimate of the pre-slide slope surface, and subtracting this surface from the present seafloor surface.

The maximum observed landslide from multibeam surveys is taken as the maximum landslide for a given region. It may be possible that larger landslides could occur in a given region, however this determination of the maximum landslide is consistent with the overall definition of PMT as "the most severe of the natural phenomena that have been historically reported or determined from geological and geophysical data for the site and surrounding area". In this case, the maximum landslide is taken from geologic observations spanning tens of thousands of years. Moreover, because landslide volumes appear to follow a power-law or log-normal distribution (ten Brink and others, 2006; Chaytor and others, 2009), there may be no mathematical or physical constraints on the definition of the theoretical maximum landslide (other than the dimensions of the entire continental slope). These calculations were only completed for part of the East Breaks landslide, the Mississippi Canyon landslide, and a landslide from the slope above the Florida Escarpment. No calculations were made for failures above the Campeche Escarpment because currently available bathymetric data are inadequate.

East Breaks Landslide

Geologic Setting: River delta that formed at the shelf edge during the early Holocene

Post Failure Sedimentation: Landslide source area appears to be partially filled (predominantly failure deposits with some post-failure sedimentation)

Age: 10,000 – 25,000 years (Piper, 1997; Piper and Behrens, 2003)

Maximum Single Event (East Breaks landslide): Maximum and minimum parameters are taken from different interpretations of the digitized failure scar surrounding the excavation region (Chaytor and others, 2009).

Volume	Area	Width	Length	Excavation Depth	Runout Distance
Max: 21.95 km^3 Min: 20.80 km^3	519.52 km^2 420.98 km^2	~ 12 km	~ 50 km	~160 m	91 km

Run out distance: 91 km from end of excavation and 130 km from headwall based on GLORIA mapping (Rothwell and others, 1991) (See FSAR Figure 2.4.6.4.3-7). Multibeam bathymetry is not available for the entire run-out area

Trabant and others (2001) have reported volumes of 50-60 km^3 and a run-out distance of 160 km. Trabant and others (2001) derived their volume estimate from the size of debris lobes in the deposition region, using a 3D seismic reflection dataset that is proprietary. The staff cannot confirm their result for that reason and because we lack the necessary bathymetry coverage that far downslope to identify the extent of the debris lobes. Debris lobes are often the result of multiple events that are difficult to distinguish (Chaytor and others, 2009; Twichell and others, 2009) and may include sediment entrainment during flow. Our volume estimate above is for the amount excavated at the source (within the landslide scarp) and is more representative of a single failure event.

Mississippi Canyon

Geologic Setting: River delta and fan system

Age: 7,500 to 11,000 years (Coleman and others, 1983; Chapter 3 in ten Brink and others, 2007)

Maximum Single Event

Volume	Area	Excavation Depth	Runout Distance
425.54 km^3	3687.26 km^2	~300 m	297 km

Other reported volumes are1500-2000 km^3 (Coleman and others, 1983). As with the East Breaks landslide, this estimate is from landslide deposits that most likely represent multiple failure episodes. The volume given above is the staff's best estimate of a maximum single-event volume.

Florida Escarpment and Slope

Geologic Setting: The slope above the edge of a carbonate platform

Post Failure Sedimentation: None visible on multibeam images or on available high-resolution seismic profiles (Twichell and others, 1993).

Age: Early Holocene or older (Doyle and Holmes, 1985). Because the deposits from these carbonate failures accumulate along the base of the Florida escarpment are buried by Mississippi Fan deposits, they are older than the youngest fan deposits dated at about 11,500 years old.

Maximum Single Event

Volume	Area	Excavation Depth	Runout Distance
16.2 km^3	647.57 km^2	~150 m but quite variable	Uncertain.

Runout distance: The landslide deposit is at the base of the Florida Escarpment buried under younger Mississippi Fan deposits.

Campeche Escarpment

Geologic Setting: Carbonate platform

One of the persistent issues during the independent confirmatory analysis is acquiring sufficient geologic information about the Campeche Escarpment with which to estimate the maximum landslide parameters as with the other Gulf of Mexico landslide provinces. Plans to conduct multibeam bathymetry surveys are pending. Presently, there is no published information showing the detailed bathymetry or distribution of landslides on or above the Campeche Escarpment.

Seismic Seiches

Seismic seiches are fundamentally a different type of wave than tsunamis. Rather than being impulsively generated by displacement of the sea floor, seismic seiches occur from resonance of seismic surface waves (continental Rayleigh and Love waves) within enclosed or semi-enclosed bodies of water. The harmonic periods of the oscillation are dependent on the dimensions and geometry of the body of water. In 1964, seiches were set up along the Gulf Coast from seismic surface waves emanating from the M=9.2 Gulf of Alaska earthquake. The efficiency at which the seiches occurred at great distance from the earthquake is primarily explained by amplification of surface wave motion from the thick sedimentary section along the Gulf Coast (McGarr, 1965). Because the propagation path from Alaska to the Gulf Coast is almost completely continental (McGarr, 1965) and because the magnitude of the 1964 earthquake is close to the maximum possible for that subduction zone (e.g., Bird and Kagan, 2004), it is likely that the historical observations of 1964 seiche wave heights are the maximum possible and less than the PMT amplitudes from landslide sources.

In summary, the NRC Staff list the following findings of our independent confirmatory analysis of the tsunami source characteristics:

- There is sufficient evidence to consider submarine landslides in the Gulf of Mexico as a present-day tsunami hazard for the purpose of defining the PMT at the Levy County Site.

- Four landslide provinces are defined in the Gulf of Mexico that are applicable for determining the PMT: Northwest Gulf of Mexico, Mississippi Canyon, slope above the Florida Escarpment, and Campeche Escarpment.

- Parameters for the maximum submarine landslide were determined for each of the provinces, except for the Campeche Escarpment where we are awaiting additional data.

- It is likely that seismic seiche waves resulting from the 1964 Gulf of Alaska earthquake are nearly the highest possible, owing to a predominantly continental ray path for seismic surface waves from Alaska to the Gulf Coast, However, they are smaller than the PMT amplitudes from submarine landslides in the region.

2.4.6.4.4 Tsunami Analysis

Information Submitted by the Applicant

The applicant's tsunami analysis primarily consists of using past studies to ascertain the tsunami propagation characteristics from the three source regions discussed in Section 2.4.6.3 to estimate tsunami amplitudes offshore of the Levy County Nuclear Plant site. Different types of tsunami analyses were used to estimate tsunami water levels for each of the three source regions.

For tsunami sources located in the far-field, the applicant only considers a source with characteristics similar to the 1755 Lisbon tsunami in their tsunami analysis. To determine tsunami amplitudes in the Gulf of Mexico from this far-field earthquake, the applicant cites the results of Mader (2001). The applicant indicates that Mader (2001) uses the nonlinear long wave equations and a 10-minute bathymetric grid to calculate tsunami amplitudes.

For tsunami sources located in the Caribbean region, the applicant cites analysis of open-ocean propagation presented by Knight (2006) (FSAR reference 2.4.6-225) and the USGS Administrative Report (2007) describing tsunami sources affecting U.S. Atlantic and Gulf Coasts (FSAR reference 2.4.6-214). The tsunami analysis method used by Knight (2006) is not indicated by the applicant. The Caribbean sources used in the analysis by Knight (2006) include earthquakes along the northern Caribbean subduction zone (i.e., the "Puerto Rico Trench" as termed by Knight, 2006), a source possibly related to the Cayman transform fault system (i.e., the "Swan fault" offshore Cancun, Mexico as termed by Knight, 2006), and the northern South America convergence zone (incorrectly called the "North Panama Deformed Belt" by Knight (2006) and by the applicant). The tsunami analysis method used in the USGS Administrate Report (2007) is a finite-difference approximation to the linear-long wave equations. Tsunami propagation across the continental shelf and tsunami runup were not modeled in this study. The Caribbean sources used in the USGS (2007) analysis as indicated by the applicant include earthquakes along the northern Caribbean subduction zone, the Cayman transform fault system, the North Panama Deformation Belt, and the northern South America convergence zone.

For tsunami sources located in the Gulf of Mexico region, the applicant considers both earthquake and landslide sources. Although intra-plate sources in the vicinity of the Mw=5.8 September 10, 2006 Gulf of Mexico earthquake are not further considered for tsunami analysis by the applicant, an offshore Veracruz tsunami scenario from Knight (2006) is considered, which the applicant links to intra-plate seismicity. As with the Caribbean tsunami sources where the applicant cites the work of Knight (2006), the applicant does not indicate the tsunami analysis method used for the Veracruz tsunami scenario. For landslide sources in the Gulf of Mexico, the applicant uses a tsunami attenuation function (FSAR equation 2.4.6-1) derived by Zahibo et al. (2003) (FSAR reference 2.4.6-222) for tsunamis originating in the Caribbean region. The theoretical basis for this attenuation function and evidence of its applicability for tsunamis in the Gulf of Mexico is not included in the FSAR. The applicant uses a Monte Carlo analysis to establish the maximum wave height near the Levy County Nuclear Plant from this attenuation function.

In order to obtain a complete description of the analysis procedure used to calculate tsunami wave height and period at the site, including the theoretical bases of the models, including the applicant's verification and the conservatism of all input parameters, the staff issued RAIs 02.04.06-08 and 02.04.06-09. In RAI 02.04.06-08, the staff asked the applicant to provide theoretical basis, assumptions (e.g., source parameterization), and applicability to the Levy County site for the tsunami attenuation function discussed on pg. 2.4-53 (Equation 2.4.6-1) and make available the details of the Monte Carlo analysis used to estimate the maximum wave height and where the maximum wave height estimate is geographically located. In addition, for this and other methods of tsunami analysis indicated in the FSAR, provide the procedure use to calculate tsunami propagation, runup, and inundation (i.e., tsunami water levels) at the Levy County site from offshore tsunami amplitude.

The applicant responded to the staff's RAI 02.04.08 in letters dated July 22, 2009 (ML0920800771) and August 10, 2010 (ML1022900851). The applicant provided a substantial new effort regarding analysis for tsunami generation, propagation, and runup. However, there are several unresolved issues in the applicant's response: (1) the formulas for source amplitude are poorly documented (they are not contained in Silver et al., 2009); (2) water depths listed in Table 1 seem arbitrary (its 300-800 m for East Breaks); (3) it is unclear how source "diameter" is determined; (4) there are typographic errors in the numbers for the Veracruz and Venezuela source diameters (Table 4); (5) the assumption that "wave amplitude onshore cannot exceed its estimated runup height at shore," is incorrect but this may be an issue with the terminology; and (6) variable *Co* in equations 17 and 18 is undefined. The applicant needs to provide additional details regarding the method for tsunami analysis in reference to the aforementioned items. In RAI 02.04.06-15, the staff requested additional information related to these six unresolved issues.

The applicant responded to the staff's RAI 02.04.06-15 in a letter dated March 25, 2010, with additional details. However, the revised equations are now incorrect, according to the most recent review article of Ward (2010). The staff issued RAI 02.04.06-16, asking the applicant to provide additional details regarding the new methodology for tsunami analysis described in response to RAI 02.4.06-08 and RAI 02.04.06-15. This discussion should specifically include: (1) the basis for source amplitude formulae; (2) clarify what is meant by "wave amplitude onshore cannot exceed its estimated runup height at shore" (statement is incorrect using standard tsunami terminology); and (3) definition of variable C_o in equations 17 and 18. The applicant responded to the staff's RAI 02.04.06-16 in a letter dated November 30, 2010 (ML1034206451). The application of the equations and understanding of the assumptions and approximations behind the method were still incorrect.

The staff issued RAI 02.04.06-17, asking the applicant to provide the following:

> An analysis of the PMT event using a technically sound and conservative approach such as those predicted by a site and region specific model approach applicable to tsunami waves to calculate tsunami water levels at or near the site. Such a model avoids approximations of source geometry, bathymetry between the source and offshore of site, and topography near the site inherent in the applicant's current approach. For example, shallow water wave equation models (COMCOT, ComMIT. Delft3D) and Boussinesq-type Models (COULWAVE, FUNWAVE, Geowave) for earthquake and earthquake/landslide/ impact generated tsunamis, respectively.
>
> If a numerical model is used, provide a clear presentation of all equations used, discussion of assumptions inherent in these equations and the associated conservatism, and the procedure to calculate the water-level values. Please provide all input data sources, calculation packages, and any associated modeling input files.

(a) If the existing approach which relies on the Ward et al publication is used, proper usage of these methods must be checked, and a complete presentation of the theoretical assumptions, as relevant to propagation modeling of a landslide-generated wave and runup/inundation, should be provided. The applicant must provide site-specific justification as to why the Ward (2010) equations are applicable and conservative for the Levy site. This would typically involve presenting the theoretical assumptions behind the generation, attenuation, shoaling, and runup equations, and why these assumptions are valid and conservative with respect to site-specific conditions. Specifically:

> <u>Tsunami Generation</u>: (1) Provide the reference for wave amplitude Equation 2.4.6-3, along with relevant assumptions used to develop that equation. (2) Provide references for the expressions of slide velocity and a clear indication as to which expressions were used to calculation the slide velocities listed in FSAR Table 2.4.6- 206. (3) Provide the rationale and justification for using Equation 2.4.6-8 derived for impact tsunami sources to model landslide tsunamis, particularly with regard to difference in wave characteristics between landslide and impact tsunamis. (4) Explain how diameter listed for each source in FSAR Table 2.4.6-206 relates to landslide parameters.
>
> <u>Tsunami Propagation</u>: (1) Explain how the "measurement point" is chosen to determine R, the distance of measurement point from the source. (2) Because the "measurement point" is a nearshore location, justify the use of Equation 2.4.6-11 that is derived for constant water depth, considering the broad continental shelf

offshore western Florida. (3) If in a revised procedure applicant applies the propagation and shoaling terms at the edge of the continental shelf, provide an expression for propagation across the continental shelf. (4) The equation for the attenuation curves (2.4.6-8) is miss-cited. Provide the correct reference, domain of applicability of these fitted curves, and assumptions used to derive these curves.

Tsunami Runup: (1) Definition of *h* in Equation 2.4.15 is inconsistent with the definition indicated in FSAR References 2.4.6-228 and 2.4.6-237, from which this equation was taken. In the revised FSAR, applicant indicates that *h* represents "shoreline wave height" whereas it is intended to represent runup as described in the aforementioned References. Provide clarification of the use of Equation 2.4.15. (2) Provide the theoretical assumptions behind the equation 2.4.15, and why these assumptions are valid and conservative with respect to site-specific conditions. (3) If revised Equation 2.4.15 is used to calculate runup, confirm that revised section 2.4.6.6.3.5 is not necessary. (4) Provide the geographic location (lat, long) and water depth where the shoaled amplitude A(R) in FSAR Table 2.4.6-207 is calculated. (5) Provide location information for revised figure 2.4.6-230 "Landward Topographic Profile", for example, in a map figure.

The applicant responded to the staff's RAI 02.04.06-17 in letters dated February 28, 2011, April 19, 2011, and July 14, 2011. Using the FUNWAVE-TVD tsunami model, the applicant provided a detailed, site-specific, technically sound and conservative approach to calculate tsunami propagation, runup, and inundation (i.e., tsunami water levels) at the Levy County site, including proposed FSAR revisions. Therefore, the staff considers RAI 02.04.06-08, RAI 02.04.06-15, RAI 02.04.06-16 and RAI 02.04.06-17 to be resolved.

The staff issued RAI 02.04.06-09, asking the applicant to provide clarification in the updated FSAR to resolve the inconsistency of the statement that the Gulf of Mexico contains no sources of reverse faults (1st sentence, section 2.4.6.4.1.2, pg. 2.4-52) given the mechanism of the September 10, 2006 Mw=5.8 in the NE Gulf of Mexico (third sentence). The applicant responded to the staff's RAI 02.04.09 in a letter dated July 22, 2009 (ML0920800771). The applicant clarifies that they meant to indicate that there are no subduction zone faults in the Gulf of Mexico, without adding specific explanation for the possibility of intra-plate reverse faults, such as the September 20, 2006 earthquake. Therefore, the staff considers RAI 02.04.06-09 to be resolved.

NRC Staff's Technical Evaluation

Numerical simulations of tsunami propagation have made great progress in the last thirty years. Several tsunami computational models are currently used in the National Tsunami Hazard Mitigation Program, sponsored by the National Oceanic and Atmospheric Administration, to produce tsunami inundation and evacuation maps for the states of Alaska, California, Hawaii, Oregon, and Washington. The computational models include MOST (Method Of Splitting Tsunami), developed originally by researchers at the University of Southern California (Titov and Synolakis, 1998); COMCOT (Cornell Multi-grid Coupled Tsunami Model), developed at

Cornell University (Liu and others, 1995); and TSUNAMI2, developed at Tohoko University in Japan (Imamura, 1996). All three models solve the same depth-integrated and 2D horizontal (2DH) nonlinear shallow-water (NSW) equations with different finite-difference algorithms. There are a number of other tsunami models as well, including the finite element model ADCIRC (ADvanced CIRCulation Model For Oceanic, Coastal And Estuarine Waters) (e.g., Myers and Baptista, 1995).

Earthquake generated tsunamis, with their very long wavelengths, are ideally matched with NSW for transoceanic propagation. Models such as Titov & Synolakis (1995) and Liu et al. (1995) have been shown to be reasonably accurate throughout the evolution of a tsunami, and are in widespread use today. However, when examining the tsunamis generated by submarine mass failures, the NSW can lead to significant errors (Lynett and others, 2003). The length scale of a submarine failure tends to be much less than that of an earthquake, and thus the wavelength of the created tsunami is shorter. To correctly simulate the shorter wave phenomenon, one needs equations with excellent shallow to intermediate water properties, such as the Boussinesq equations. While the Boussinesq model too has accuracy limitations on how deep (or short) the landslide can be (Lynett and Liu, 2002), it is able to simulate the majority of tsunami generating landslides. Thus, for the work proposed here, the Boussinesq-based numerical model COULWAVE (Lynett and Liu, 2002) will be used. (See Appendix for reprints of peer-reviewed papers that form the foundation of COULWAVE.) This model solves the fully nonlinear extended Boussinesq equations on a Cartesian grid. COULWAVE has the capability of accurately modeling the wind waves with both nonlinear and dispersive properties. A particular advantage of the model is the use of fully non-linear equations for both deep and shallow water. This avoids the common problem of "splitting" the analysis when the wave reaches shallow water. Applications for which COULWAVE has proven very accurate include wave evolution from intermediate depths to the shoreline, including parameterized models for wave breaking and bottom friction. For technical details on wave propagation, breaking, runup, inundation, and overtopping of sloping structures see Geist et al., (2009) (including the references).

In response to **RAI 02.04.06-17**, the applicant models a tsunami from the Mississippi Canyon landslide using a FUNWAVE. FUNWAVE is a phase-resolving, time-stepping Boussinesq model for ocean surface wave propagation in the nearshore. For confirmatory analysis, the NRC staff used a higher-order Boussinesq hydrodynamics model (COULWAVE), which is more specifically suited to landslide tsunamis. As described above, the staff considers **RAI 02.04.06-17** to be resolved**.**

2.4.6.4.5 Tsunami Water Levels

Information Submitted by the Applicant

The various methods of tsunami analysis used by the applicant to estimate tsunami water levels at the Levy County Nuclear Plant site are described at the beginning of Section 2.4.6.4.4. Most of the water level estimates are taken directly from previously published studies. The exception is the analysis for the East Breaks landslide in the Gulf of Mexico, where the applicant uses a tsunami attenuation function and Monte Carlo analysis to establish the maximum water level.

The applicant provided the following table summarizing the water level estimates for each of the sources considered:

Location	Mechanism	Magnitude	Offshore Wave Height	Estimated Runup	Validation of Source as Potential Tsunami Generator	Analysis Reference
West Cayman oceanic transform fault (also known as Swan Island fault)	Earthquake	Mw 8.35	13 cm (5.1 in)	39 cm (15.4 in)	Bird (2003)	USGS (2007)
East Cayman fault (also known as Oriente fault)	Earthquake	Mw 8.45	12 cm (4.72 in)	36 cm (14.2 in)	Bird (2003)	USGS (2007)
Northern Puerto Rico/Lesser Antilles	Earthquake	Mw 8.84	14 cm (5.5 in)	42 cm (16.5 in)	Bird (2003)	USGS (2007)
North Panama deformation belt	Earthquake	Mw 8.28	25 cm (9.8 in)	75 cm (29.5 in)	Bird (2003)	USGS (2007)
North Venezuela subduction zone	Earthquake	Mw 8.5	65 cm (25.6 in)	195 cm (76.8 in)	Bird (2003)	USGS (2007)
Puerto Rico trench (66W, 18N)	Earthquake	Mw 9.0	25 cm (9.8 in)	75 cm (29.5 in)	Bird (2003)	Knight (2006)
Caribbean Sea (85W, 21N) (translated from the Swan fault to mouth of Gulf near Cancun)	Earthquake	Mw 8.2	30 cm (11.8 in)	90 cm (35.4 in)	Bird (2003)	Knight (2006)
North Panama Deformed Belt (66W, 12N)	Earthquake	Mw 9.0	15 cm (5.9 in)	45 cm (17.7 in)	Bird (2003)	Knight (2006)
Gulf of Mexico, offshore of Veracruz (95W, 20N)	Earthquake	Mw 8.2	35 cm (13.8 in)	105 cm (41.3 in)	hypothetical	Knight (2006)
East Breaks Slump	Landslide	50 to 60 cubic kilometers (km^3)	1.68 m (5.5 ft)	5.04 m (16.5 ft)	Trabant (2001); tsunami claim not further supported	Trabant (2001), Zaibo (2003)

As indicated previously, the “North Panama Deformed Belt” is incorrectly identified by Knight (2006) and the applicant and is not the same region defined as the North Panama deformation belt by USGS (2007). Knight’s (2006) “North Panama Deformed Belt” source is geographically located along the northern South America convergence zone (also known as the north Venezuela subduction zone). The “Estimated Runup” values indicated in the applicants table above were determined by applying an amplification factor of 3 to the “Offshore Wave Height” values, as indicated by the applicant during the site audit. Not included in this table is the applicant’s Gulf of Mexico offshore wave height estimate of “less than one meter” from the 1755 Lisbon far-field seismogenic tsunami (Mader, 2001) as cited on pg. 2.4-55 of the FSAR. It is unclear whether high tide and long-term sea-level rise are included in determining these water levels.

The applicant indicates that the nominal plant grade elevation is 15.2 m (NAVD88) and therefore the water level from the Probable Maximum Tsunami will not impact safety-related facilities at the Levy County Nuclear Plant site.

In order to obtain a complete description of the ambient water levels assumed to be coincident with the tsunami, the staff issued RAI 02.04.06-10, asking the applicant to provide a discussion in the updated FSAR of the value for 10% exceedance high-tide and long-term sea-level rise coincident with maximum tsunami water levels at the Levy County site. The applicant responded to the staff’s RAI 02.04.10 in a letter dated July 22, 2009 (ML0920800771). The applicant provided details of high spring tide, sea-level anomaly and sea-level rise in the calculation of

PMT water levels. Based on the applicant's response, the staff considers RAI 02.04.06-10 to be resolved.

NRC Staff's Technical Evaluation

Numerical modeling of three different types of tsunami sources has been performed to determine their impact on the Levy County site. The three source types are: (1) distant earthquake sources; (2) a regional earthquake source in the Gulf of Mexico; and (3) regional submarine landslide sources in the Gulf of Mexico. Most of the analysis described in this section is focused on source type (3) for determination of the PMT. For all conditions, the most conservative source parameters were employed, even when arguably unphysical, to provide an absolute upper limit on the possible tsunami effects at the Levy County site.

a. Distant Earthquake Sources

Regional tsunami propagation patterns in the Gulf of Mexico have been computed for a number of distant earthquake sources located in the Caribbean as reported in ten Brink et al. (2008). In Chapter 8 of that study, earthquake scenarios along five fault systems were examined: (1) west Cayman oceanic transform fault (OTF); (2) east Cayman OTF; (3) northern Caribbean subduction zone; (4) north Panama Oceanic Convergence Boundary; and (5) the northern South America convergent zone. In that report, tsunami propagation was modeled using the leap-frog, finite-difference approximation to the linear-long wave equations computed using Cartesian coordinates. Bottom friction, wave breaking, and runup were not modeled—computations were restricted to water depths of 250 m or greater. Results for the western Gulf of Mexico indicate that offshore tsunami amplitudes were less than 1.0 m for each earthquake scenario.

For comparative purposes, we re-compute here the offshore tsunami water levels for earthquake scenarios (3) and (5) using the COMCOT model. The COMCOT model is more accurate than the model used in ten Brink et al. (2008) since it includes non-linear terms in the propagation equations (hence, the computations can be carried into shallower water than in ten Brink et al., 2008), a moving boundary condition at the shoreline, and is computed in spherical coordinates. Bottom friction is also included, but is set at a low, conservative value ($f = 10^{-4}$) in this case.

These results confirm that tsunami amplitudes from distant Caribbean earthquakes are less than 1.0 m near the Levy County site. Tsunami amplitudes from earthquakes along the Azores-Gibraltar oceanic convergence boundary are also likely to be less than 1 m in the Gulf of Mexico (Mader, 2001; Barkan and others, 2009).

b. Regional Earthquake Sources

Regional tsunami propagation patterns in the Gulf of Mexico have been computed for a local earthquake near the location of the September 10, 2006 M=6.0 earthquake. For this scenario, probable maximum fault dimensions and slip similar to an M_{max}=7.5 earthquake (Petersen and

others, 2008; Wheeler, 2009; Mueller, 2010) was determined from the empirical scaling relationships for intra-plate earthquakes of Wells and Coppersmith (1994). Conservative values were allowed within 1 standard deviation of the empirical estimates of all fault types (empirical relationships for reverse faults only are not statistically reliable). This resulted in the following rupture parameters: length=150 km; width=30 km, average slip= 5m. The corresponding magnitude, assuming a shear modulus of 30 GPa, is M_w=7.8—slightly greater than M_{max}=7.5 because of the conservative assumptions. The geometric parameters of the earthquake were taken from the nodal plane of the September 10, 2006 M=6.0 earthquake that optimized the radiation of tsunami energy toward the site: dip = 47°; strike=346°; latitude=27.3°N; longitude 86.3°W.
The offshore tsunami water levels for this local earthquake scenario was computed using the COMCOT model as described for the distant earthquake sources above. Bottom friction is also included, but is set at a low, conservative value ($f = 10^{-4}$) in this case. In general, tsunami amplitudes from the local M_w=7.8 sources are larger than the distant M~9 earthquake sources, with peak tsunami amplitudes near 1 m. These amplitudes are significantly less than the tsunami amplitudes produced by the regional submarine landslide sources described below.

c. Regional Submarine Landslide Sources

Five different landslide tsunami sources in the GOM are investigated to determined their impact at the Levy site. First, all sources are simulated as one-horizontal-dimension (1HD) transects, and thus conservatively neglect radial spreading of wave energy. Additionally, each source is simulated with a wide range of frictional coefficients, from no friction to likely in-situ friction, to provide both an upper limit and a realistic estimate of the runup. From these 1HD simulations, the Mississippi Canyon source clearly has the greatest potential to bring at large wave to the Levy site, with 1HD water elevations near the site in excess of +30 m. This source and a local Florida Shelf landslide source are chosen for additional analysis by means of two-horizontal-dimension (2HD) simulations, where radial spreading is explicitly included. Interestingly, both of these sources predict a wave of similar maximum elevation at the 30 m depth offshore of the site, approximately 7 m. However, the Mississippi Canyon wave is longer in period and has a longer train of large waves, and thus is designated as the PMT for the Levy site. Highly refined nearshore simulations show that this source, even when including high tide and future sea level rise, does not produce a tsunami that reaches the Levy site ground elevation.

Numerical Grid Development

The bathymetry/topography grid required by the hydrodynamic model is created via three main sources: 1) the Smith and Sandwell (SS) 2-minute global elevation database; 2) a recent GOM grid created by the U.S. Army Corps of Engineers for use with the storm surge model ADCIRC; and 3) a blend of available bathymetry and topography for the west coast of Florida. Sources 2) and 3) are a combination of numerous databases including recent lidar surveys and digitized elevation maps. These two sources were used for bathymetry and topography at locations with bottom elevations greater than -500 m. For depths greater than this (or elevations lower), the SS was primarily used.

Figure 2.4.6.4.5-1 shows the entire GOM grid coverage, with the five tsunami landslide source locations outlined. The high level of detail in the full resolution image is not evident in this reproduced image, but the staff's review addressed the detailed GOM grid.

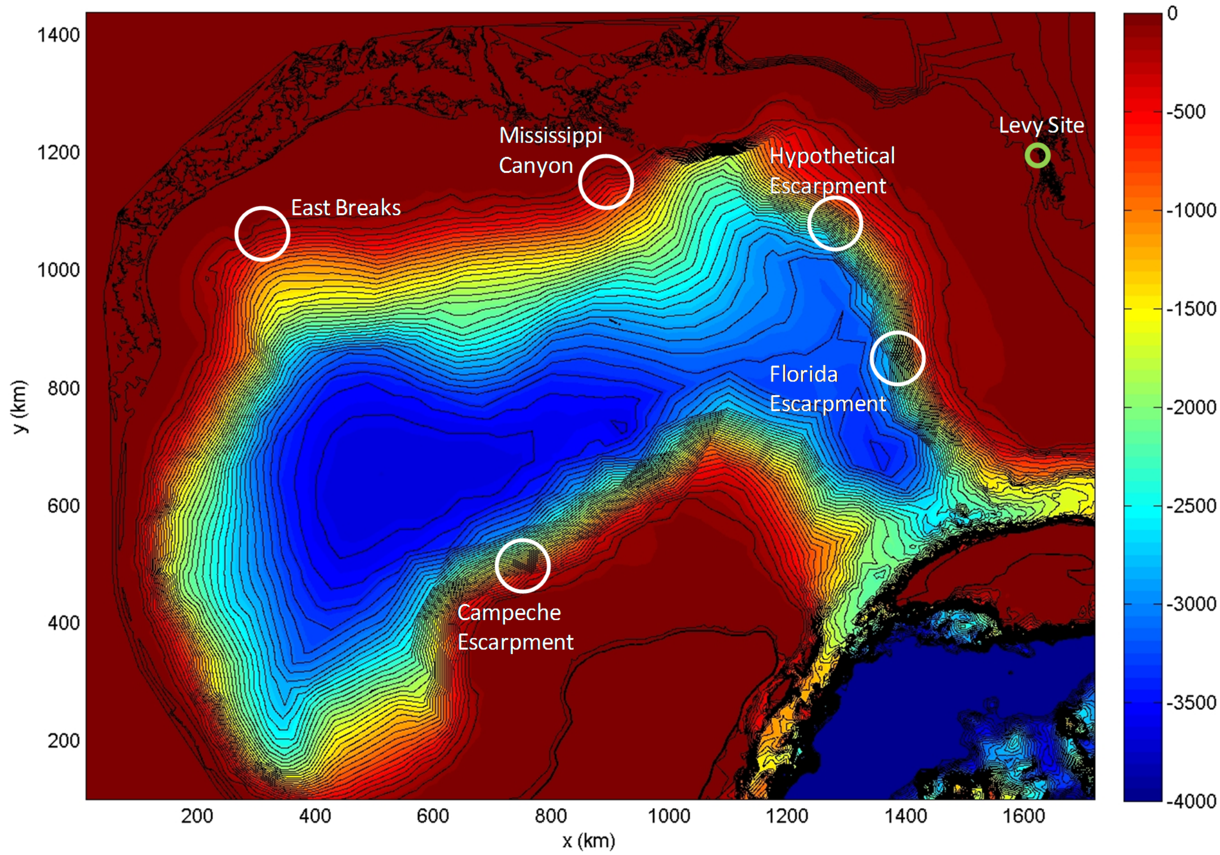


Figure 2.4.6.4.5-1. Bathymetry/topography contour surface of the GOM domain used for the tsunami hydrodynamic modeling. General locations of the five potential tsunami sources are shown by the white circles and the Levy site by the green circle. Bottom elevations are indicated by colors following the colorbar, with units in meters.

<u>Initial Numerical Simulations – Physical Limits</u>

The purpose of these initial simulations is to provide an absolute upper limit of the tsunami wave height that could be generated by the potential tsunami sources. Note that these limiting simulations use physical assumptions that are arguably unreasonable; the results of these simulations will be used to filter out tsunami sources that are incapable of adversely impacting the Levy site under even the most conservative assumptions. Specifically, these assumptions are:

1. Time scale of the seafloor motion is very small compared the period of the generated water wave (tsunami)
2. Bottom roughness, and the associated energy dissipation, is negligible in locations that are initially wet (i.e. locations with negative bottom elevation, offshore)

Assumption 1 simplifies the numerical analysis considerably. With this assumption, the free water surface response matches the change in the seafloor profile exactly. This type of approximation is used commonly for subduction-earthquake-generated tsunamis, but is known to be very conservative for landslide tsunamis (Lynett & Liu, 2002). The modeling simplification arises because need to include the landslide time evolution is removed. The initial pre-landslide bathymetry profile, as estimated by examination of neighboring depth contours, is subtracted by the post (existing) landslide bathymetry profile. This difference surface is smoothed and then used directly as a “hot-start” initial free surface condition in the hydrodynamic model.

Assumption 2 does not simplify the analysis significantly; however it does prevent the use of an overly high bottom roughness coefficient, which could artificially reduce the tsunami energy reaching the shoreline. Note that while the offshore regions are assumed to be without bottom friction, such an assumption is too physically unrealistic to accept for the inland regions where the roughness height may be the same order as the flow depth. For tsunami inundation, particularly for regions such as this project location where the wave would need to inundate long reaches of densely vegetated land to reach the site, inclusion of some measure of bottom roughness is necessary.

If any of these initial simulations indicate the need for more precise description of the source motion, such will be incorporated into a subsequent analysis. Source physics description and modeled motion will be given only if needed for this analysis. The most likely reason for needed higher precision would be if one of the initial simulation shows flooding at the site in exceedance of the PMF elevation determined elsewhere.

One-Horizontal Dimension (Transect) Simulations

First, one-horizontal-dimension (1HD) simulations are performed for all potential sources. The 1HD simulations require a small fraction of the CPU time of the 2HD runs, but do not include the radial spreading and refraction effects. Lack of radial spreading will lead to a conservative result in 1HD, while refraction can be either a constructive or destructive effect on the wave height, depending on the shallow water depth contours. 1HD simulations will provide an upper limit on the inundation distance and information on the relative importance of overland bottom friction, while the 2HD simulations provide insight into radial spreading and refraction. Results from the 1HD simulations will be used to filter all the sources down to a few possible candidates for the PMT; then a 2HD simulation will be run for each of these candidates.

East Breaks Landslide Source:

As provided in the landslide characterization section, the excavation depth of this slide is approximately 160 m. This length provides the trough elevation (i.e. -160 m) of the hot-start initial water surface condition. The horizontal dimensions of the slide source region are ~12 km in width and 50 km in length. With this information and knowledge of characteristic slide-generated waves taken from the literature (Lynett & Liu, 2002; Lynett & Liu, 2005), the hot-start initial condition is constructed.

<u>1HD Results (No friction)</u>: The depth transect is taken from the source location directly to the Levy site. A constant spatial grid size of 200 m is used across the transect for the 1HD cases. Predictions from three 1HD simulations are given for **A)** no bottom friction, **B)** bottom friction due to moderate roughness characteristic of grass/turf (f=0.01), and **C)** bottom friction due to large roughness characteristic of the trees and dense shrub-like vegetation currently existing seaward of the Levy site (f=0.05). Note that the three different bottom friction values are only applied over initially dry land; for all simulations the initially submerged portions of the transect use no bottom friction.

In model simulations, the offshore evolution of the East Breaks wave can be seen with clearly dispersive effects, as shown by the long train of waves that reaches the Florida shelf. All of the simulations provide identical results for the tsunami prior to reaching the shoreline, as all the simulations start with the same wave, use the same bathymetry, and are frictionless offshore. This is most evident as the tsunami approaching the site.

<u>1HD Results (Friction)</u>: As the wave starts inundating dry land, friction becomes important. The no-friction case A) shows a fast moving bore front that easily reaches the Levy site ground elevation, with maximum water surface elevations approaching +25 m at the site. Despite the modest friction value used in case B), here the tsunami wave front is slowed significantly but does reach the site, and maximum water elevations at the site are approximately +22 m. Finally, for case C), the large, realistic friction retards the flow considerably, and the tsunami wave front is stopped 3 km seaward of the site. Note that in all these figures, the horizontal and vertical scales are distorted, and that the realistic friction tsunami case still does manage to travel 15 km inland. A conclusion of this 1HD East Breaks study is that a tsunami approaching the site, with a bore height up to +12 m at the still water shoreline, will not adversely impact the site if the vegetation roughness is properly accounted for.

<u>Campeche Landslide Source:</u>

As noted in the landslide description section, there is no available data with which to constrain this source. In the absence of any quantitative guidance, it is assumed that a slide in this region will share geometric properties with the slope above the Florida Escarpment. As provided in the landslide characterization section, the excavation depth of this slide is approximately 150 m. This length provides the trough elevation (i.e. -150 m) of the hot-start initial water surface condition. The horizontal dimensions of the slide source region are assumed to be ~20 km in width and 50 km in length, inferred from the various scarps visible in the multibeam bathymetric data. With this information and knowledge of characteristic slide-generated waves taken from the literature (Lynett & Liu, 2002; Lynett & Liu, 2005), the hot-start initial condition is constructed.

1HD Results (No friction): The depth transect is taken from the source location directly to the Levy site. A constant spatial grid size of 200 m is used across the transect for the 1HD cases.

Predictions from three 1HD simulations are given for **A)** no bottom friction, **B)** bottom friction due to moderate roughness characteristic of grass/turf (f=0.01) , and **C)** bottom friction due to large roughness characteristic of the trees and dense shrub-like vegetation currently existing seaward of the Levy site (f=0.05). Note that the three different bottom friction values are only applied over initially dry land; for all simulations the initially submerged portions of the transect use no bottom friction.

In model simulations, the offshore evolution of the Campeche wave can be seen with clearly dispersive effects as shown by the long train of waves that reaches the Florida shelf. All of the simulations provide identical results for the tsunami prior to reaching the shoreline, as all the simulations start with the same wave, use the same bathymetry, and are frictionless offshore.

<u>1HD Results (Friction)</u>: As the wave starts inundating dry land, friction becomes important and the results of the three simulations diverge. The no-friction case A) shows a fast moving bore front that easily reaches the Levy site ground elevation, with maximum water surface elevations approaching +23 m at the site. Despite the modest friction value used in case B), the tsunami wave front is slowed significantly but does reach the site, and maximum water elevations at the site are approximately +14 m. Finally, for case C), the large, realistic friction retards the flow considerably, and the tsunami wave front is stopped 15 km seaward of the site. Note that in all these figures, the horizontal and vertical scales are distorted, and that the realistic friction tsunami case still does manage to travel 15 km inland. A conclusion of this 1HD Campeche study is that a tsunami approaching the site, with a bore height up to +14 m at the still water shoreline, will not adversely impact the site if the vegetation roughness is properly accounted for.

<u>Florida Slope Landslide Source:</u>

As provided in the landslide characterization section, the excavation depth of this slide is approximately 150 m. This length provides the trough elevation (i.e. -150 m) of the hot-start initial water surface condition. The horizontal dimensions of the slide source region are assumed to be ~20 km in width and 50 km in length, inferred from the various scarps visible in the multibeam bathymetric data. With this information and knowledge of characteristic slide-generated waves taken from the literature (Lynett & Liu, 2002; Lynett & Liu, 2005), the hot-start initial condition is constructed.

<u>1HD Results (No Friction)</u>: The depth transect is taken from the source location directly to the Levy site. A constant spatial grid size of 200 m is used across the transect for the 1HD cases. Predictions from three 1HD simulations are given for A) no bottom friction, B) bottom friction due to moderate roughness characteristic of grass/turf (f=0.01), and **C)** bottom friction due to large roughness characteristic of the trees and dense shrub-like vegetation currently existing seaward of the Levy site (f=0.05). Note that the three different bottom friction values are only applied over initially dry land; for all simulations the initially submerged portions of the transect use no bottom friction.

In the staff simulations, the large, nonlinear wave immediately steepens and forms a bore-front once on the shallow shelf. All of the simulations provide identical results for the tsunami prior to reaching the shoreline, as all the simulations start with the same wave, use the same bathymetry, and are frictionless offshore.

1HD Results (Friction): As the wave starts inundating dry land, friction becomes important and the results of the three simulations diverge. The no-friction case A) shows a fast moving bore front that barely reaches the Levy site ground elevation, with maximum water surface elevations approaching +14 m at the site. With the modest friction value used in case B), the tsunami wave front is slowed significantly and does not reach the site. Finally, for case C), the large, realistic friction retards the flow considerably, and the tsunami wave front is stopped 25 km seaward of the site. A conclusion of this 1HD Florida Slope study is that a tsunami approaching the site, with a bore height up to +6 m at the still water shoreline, will not adversely impact the site if the vegetation roughness is properly accounted for.

It should also be noted that one of the reasons for the relatively small wave height produced by this source, as compared to the Campeche source, is the longer length of shelf that the wave must travel over before reaching the shoreline. With the Florida Slope transect, the shelf length is 150 km longer than that for the Campeche source. A second reason for a smaller tsunami, again as compared to Campeche, is the wave orientation. For a slide on the Florida shelf, the wave approaching Florida would have a leading depression. For a slide coming from Campeche, the wave approaching Florida would have a leading elevation. Once a leading depression wave is on the shelf, nonlinear effects will cause the trailing elevation wave to overrun and partially absorb the depression, equating to a decrease in the absolute elevation of the elevation wave front.

Florida Slope WORST CASE Landslide Source:

As mentioned in the previous Florida Slope section, the very long shelf length required by drawing the transect from the existing landslide source to the site might diminish the tsunami impacts considerably. In the section, a landslide source, identical to the Florida Slope, is hypothesized to exist immediately offshore of the Levy site. By minimizing the travel time to the coast and time over the shallow shelf, this simulation will provide an upper limit of the tsunami impact at the Levy site due to a Florida Slope-type slide anywhere along the west Florida shelf.

As provided in the landslide characterization section, the excavation depth of this slide is approximately 150 m. This length provides the trough elevation (i.e. -150 m) of the hot-start initial water surface condition. The horizontal dimensions of the slide source region are assumed to be ~20 km in width and 50 km in length, inferred from the various scarps visible in the multibeam bathymetric data. With this information and knowledge of characteristic slide-generated waves taken from the literature (Lynett & Liu, 2002; Lynett & Liu, 2005), the hot-start initial condition is constructed.

1HD Results (No Friction): The depth transect is taken from the source location directly to the Levy site. A constant spatial grid size of 200 m is used across the transect for the 1HD cases. Predictions from three 1HD simulations are given for **A)** no bottom friction, **B)** bottom friction due to moderate roughness characteristic of grass/turf (f=0.01), and **C)** bottom friction due to large roughness characteristic of the trees and dense shrub-like vegetation currently existing seaward of the Levy site (f=0.05). Note that the three different bottom friction values are only applied over initially dry land; for all simulations the initially submerged portions of the transect use no bottom friction.

In the offshore evolution of the Florida Slope wave, the large, nonlinear wave immediately steepens and forms a bore-front once on the shallow shelf. All of the simulations provide identical results for the tsunami prior to reaching the shoreline, as all the simulations start with the same wave, use the same bathymetry, and are frictionless offshore.

1HD Results (Friction): As the wave starts inundating dry land, friction becomes important and the results of the three simulations diverge. The no-friction case A) shows a fast moving bore front that reaches the Levy site ground elevation, with maximum water surface elevations approaching +15 m at the site. With the modest friction value used in case B), the tsunami wave front is slowed significantly and does not reach the site. Finally, for case C), the large, realistic friction retards the flow considerably, and the tsunami wave front is stopped 15 km seaward of the site. A conclusion of this 1HD Florida Slope WORST CASE study is that a tsunami approaching the site, with a bore height up to +9 m at the still water shoreline, will not adversely impact the site if the vegetation roughness is properly accounted for. Despite the 50 percent larger nearshore wave elevation from the Florida Slope WORST CASE, as compared to the Florida Slope, the impact at the Levy site is not considerably different.

Mississippi Canyon Landslide Source:

As provided in the landslide characterization section, the excavation depth of this slide is approximately 300 m. However, this excavation, in the upper canyon, occurs near the shelf break, where the water depths away from the scarp are ~150 m. Thus the initial depression is set to the water depth at the head of the scarp, 150 m. The horizontal dimensions of the slide source region are assumed to be ~30 km in width and 160 km in length, inferred from the multibeam bathymetric data. With this information and knowledge of characteristic slide-generated waves taken from the literature (Lynett & Liu, 2002; Lynett & Liu, 2005), the hot-start initial condition is constructed.

1HD Results (No Friction): The depth transect is taken from the source location directly to the Levy site. A constant spatial grid size of 200 m is used across the transect for the 1HD cases. Predictions from three 1HD simulations are given for **A)** no bottom friction, **B)** bottom friction due to moderate roughness characteristic of grass/turf (f=0.01), and **C)** bottom friction due to large roughness characteristic of the trees and dense shrub-like vegetation currently existing seaward of the Levy site (f=0.05). Note that the three different bottom friction values are only applied over initially dry land; for all simulations the initially submerged portions of the transect use no bottom friction.

In the offshore evolution of the Florida Slope wave the large, nonlinear wave immediately steepens and forms a bore-front once on the shallow shelf. All of the simulations provide identical results for the tsunami prior to reaching the shoreline, as all the simulations start with the same wave, use the same bathymetry, and are frictionless offshore.

<u>1HD Results (Friction)</u>: The no-friction case A) shows a fast moving bore front that easily reaches the Levy site ground elevation, with maximum water surface elevations approaching +40 m at the site. Even with the modest friction value used in case B), the tsunami wave front is not slowed significantly and also easily reaches the site with water elevations of +33 m. Finally, for case C), the large, realistic friction retards the flow considerably, but still, the tsunami reaches the site, although the site is near the inundation limit. A conclusion of this 1HD Mississippi Canyon study is that a tsunami approaching the site, with a bore height up to +20 m at the still water shoreline, may impact the site. A more detailed, 2HD analysis of this site is clearly needed.

Two-Horizontal Dimension Simulations

From the 1HD simulations, it is possible to reduce the number of tsunami sources that need additional attention. The Mississippi Canyon source gives the largest heights at the shoreline, twice as large as the nearest source, and is also the closest non-Florida slope source to the site, so radial spreading effects should also be relatively minor for Mississippi Canyon. Thus, it can be reasonable expected that, if detailed 2HD simulation show that the Mississippi Canyon source has no impact at the site, then all other non-Florida slope sources (East Breaks, Campeche) can also be eliminated.

While it is likely that elimination of the Mississippi Canyon source as impacting the Levy site would also eliminate the Florida Slope WORST CASE source, because the Florida Slope WORST CASE is on the immediate shelf, radial spreading effects may not act to decrease the incoming wave height significantly. 2HD wave heights may be quite similar to those predicted by the 1HD simulation, which showed the tsunami reaching the site for the no-friction case. Therefore, two sources, Mississippi Canyon and Florida Slope WORST CASE, are discussed further in this SER.

Florida Slope WORST CASE Landslide Source

The slide and initial water surface condition properties for this source are described above in the corresponding 1HD section, but are given again here for completeness. As provided in the landslide characterization section, the excavation depth of this slide is approximately 150 m. This length provides the trough elevation (i.e. -150 m) of the hot-start initial water surface condition. The horizontal dimensions of the slide source region are assumed to be ~20 km in width and 50 km in length, inferred from the various scarps visible in the multibeam bathymetric data. With this information, and knowledge of characteristic slide-generated waves taken from the literature (Lynett & Liu, 2002; Lynett & Liu, 2005), the hot-start initial condition is constructed. A constant spatial grid size of 500 m is used in the numerical simulation.

The 2HD evolution, within 15 minutes from the landslide, it is clear that radial spreading effects are important offshore of the shelf, but on the shelf, where the wave is approaching the Levy site, this is not the case. Spreading is minor, and the wave energy remains in a laterally compact front. The elevation component of the landward traveling wave forms into a bore about 30 minutes after the slide and quickly overtakes the leading depression. The bore front height continues to diminish and by the time the front reaches a depth of about 30 m its elevation is approximately 7 m. Note that for the 1HD simulation, the wave height at this depth was 10 m, a relatively minor reduction. Results from this simulation will be analyzed further and compared with the 2HD Mississippi Canyon results in a later section.

Mississippi Canyon Landslide Source

The slide and initial water surface condition properties for this source are described above in the corresponding 1HD section, but are given again here for completeness. The initial depression is set to the water depth at the head of the scarp, 150 m. The horizontal dimensions of the slide source region are assumed to be ~30 km in width and 160 km in length, inferred from the multibeam bathymetric data. With this information and knowledge of characteristic slide-generated waves taken from the literature (Lynett & Liu, 2002; Lynett & Liu, 2005), the hot-start initial condition is constructed. A constant spatial grid size of 500 m is used in the numerical simulation.

In the 2HD evolution, within 20 minutes from the landslide, it is clear that radial spreading effects are important for the wave approaching the site. By the time the wave has reached the shelf break the leading elevation wave height is ~15 m, a significant reduction from the hot start elevation of 120 m. The elevation component of the landward traveling wave forms into a bore once on the shelf. The bore front height continues to diminish, and by the time the front reaches a depth of about 30 m, its elevation is approximately 7 m. Note that for the 1HD simulation, the wave height at this depth was 25 m.

<u>Local Evolution of the Tsunami in the Nearshore Areas of the Site</u>

Finally, propagation over the shallow, nearshore bathymetry at the site is examined. The purpose of these simulations is to provide very refined 2HD inundation using the best available bathymetry and topography near the site. This subdomain is nested inside the large-scale 2HD domains discussed above for the Florida Slope WORST CASE and Mississippi Canyon sources. The offshore boundary, situated at a depth of 30 m, is forced with results from the large-scale 2HD simulations. Interestingly, the peak elevations of the wave trains are nearly identical, with the peak Mississippi Canyon crest elevation of 7.2 m, and the peak Florida Slope WORST CASE crest elevation of 6.9 m. The periods of the wave components in these two wave trains are slightly different, with the period from the Mississippi Canyon source at 45 minutes and that from the Florida Slope WORST CASE at 38 minutes. The most significant difference between the two trains is the number of large waves in the train. The Mississippi Canyon wave train has four distinct waves with crest elevation greater than 2 m, while the Florida Slope WORST CASE train has just one. With these comparisons in mind, is it evident that the Mississippi Canyon source produces the PMT for this site, and will be the only source used to simulate the refined, nearshore tsunami impact.

A subdomain, approximately 200 km by 150 km, centered 75 km offshore is used here.. A constant grid size of 100 m is used, and both the seafloor and initially dry land is assumed smooth, with no bottom friction dissipation. This is the most conservative assumption, and provides an upper physical limit for the inundation distance. As mentioned above, the offshore boundary is forced with the Mississippi Canyon sea surface time series. The interaction with the coastline is complex, owing to the complex bathymetry and topography, and the runup elevation is highly variable across the shoreline. In the lower (southern) part of the domain, where relatively steep topography is located close to the shoreline, the maximum runup elevation is +8 m and the inundation distance is ~ 8 km. However, immediately seaward of the site, where a wide, coastal plan exists, the runup elevation is +3 m, but the inundation distance is ~18 km. Thus, the tsunami does not come close to the site ground elevation.

The above simulation assumes that the tsunami event occurs at mid-tide with current sea levels. Independent analysis of the 10% exceedance high tide was conducted for 16 years of NOAA NOS CO-OPS data at the Clearwater Beach, FL tide gauge station (years 1973-2006). The 10 percent exceedance high tide was determined to be 0.75 m (NAVD88) for these years, compared to 0.82 m indicated in the applicant's response to RAI 2.4.6-10. The long-term sea-level rise at the Clearwater Beach, FL station is 2.43±0.80 mm/yr according to NOAA NOS-CO-OPS data. Therefore our estimated antecedent water level is 0.75 m (high tide) + 0.18 m (sea level anomaly) + 0.32 m (100-year sea level rise + 1s.d.) = 1.2 m (NAVD88). The applicant's estimated antecedent water level is 1.1 m (NAVD88) as indicated in their response to RAI 2.4.6-10.

A final simulation, using the identical numerical configuration described in the preceding paragraph is run, with the higher water levels. The maximum runup offshore of the site, using the water level increased by 1.2 m, is +6.1 m. Thus, by increasing the water depth by 1.2 m, the runup elevation was increased by 3.1 m. Clearly, the process of bore evolution is highly nonlinear, and the increase in the water depth allows for a measurably larger wave to reach the shoreline and push farther inland than would be expected by a simple linear addition of the water depth increase (1.2 m) to the previous runup prediction (+3.0 m). However, even when considering this, the maximum tsunami runup in the vicinity of the site does not approach the Levy site ground elevation.

Summary

Numerical modeling of three different types of tsunami sources has been performed to determine their impact on the Levy County site. The three source types are (1) distant earthquake sources; (2) a regional earthquake source in the Gulf of Mexico; and (3) regional submarine landslide sources in the Gulf of Mexico. For the latter source type that defines source for the PMT, water levels from five different submarine landslide scenarios were calculated using COULWAVE to determine the PMT.

Using conservative source parameters and neglecting the radial spreading of wave energy, the 1HD simulations indicate that the Mississippi Canyon source clearly has the greatest potential to bring a large wave to the Levy site, with 1HD water elevations near the site in excess of +30 m. 2HD simulations of this source and the WORST CASE Florida Slope landslide source that include radial spreading predict a maximum wave elevation of 7 m offshore of the site (30 m water depth). However, the Mississippi Canyon wave is longer in period and has a longer train of large waves, and thus is designated as the PMT for the Levy site. Highly refined nearshore simulations show that this source results in a maximum water level of +3 m. Because of nonlinear effects during wave propagation, one cannot simply add an antecedent sea level that includes 10% exceedance high tide, sea level anomaly, and sea-level rise to this maximum water to the +3m maximum water level. A separate simulation that includes the nonlinear propagation effects and a +1.2 m (NAVD88) antecedent sea level results in a maximum water level of +6.1 m. Thus, the PMT does not reach the Levy site plant grade elevation.

2.4.6.4.6 Hydrography And Harbor Or Breakwater Influences On Tsunami

Information Submitted by the Applicant

The applicant indicates that routing of the controlling tsunami, including breaking wave formation and resonance effects, is expected to be minor and limited to shorelines. In addition, the applicant indicates that hydrography and harbor or breakwater influences are not expected to be severe enough to impact safety-related structures.

NRC Staff's Technical Evaluation

The NRC Staff concurs with the applicant in that the hydrography and harbor or breakwater influences are not expected to be severe enough to impact safety-related structures. The offshore hydrography and harbor or breakwater influences are specifically accounted for in the numerical modeling performed during the independent confirmatory analysis.

2.4.6.4.7 Effects On Safety-Related Facilities

Information Submitted by the Applicant

The applicant indicates that the effects of the Probable Maximum Tsunami are not expected to be severe enough to impact the operation of safety-related structures. The applicant further indicates that measures to protect the site against the effects of tsunami are not included in the design criteria.

NRC Staff's Technical Evaluation

The NRC Staff concurs with the applicant in that the effects of the Probable Maximum Tsunami are not expected to be severe enough to impact the operation of safety-related structures

2.4.6.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.6.6 *Conclusion*

The staff reviewed the COL application and confirmed that the COL applicant has addressed the information relevant to design basis for tsunami flooding. The staff also confirmed that there is no outstanding information required to be addressed in the COL FSAR related to this section.

The staff reviewed the information provided and, for the reasons given above, concludes that the COL applicant has provided sufficient details about the site description to allow a staff evaluation, as documented in Section 2.4.6 of this report. Based on the above, the staff concludes that the identified site characteristics meet the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.20(c), with respect to establishing the design basis for SSCs important to safety. The information addressing the COL Information Item 2.4.6 is adequate and acceptable.

2.4.7 Ice Effects

2.4.7.1 *Introduction*

FSAR Section 2.4.7 addresses ice effects to ensure that safety-related facilities and water supply are not affected by ice-induced hazards.

Section 2.4.7 of this SER presents an evaluation of the following topics based on data provided by the applicant in the FSAR and information available from other sources: (1) regional history and types of historical ice accumulations (i.e., ice jams, wind-driven ice ridges, floes, frazil ice formation, etc.); (2) potential effects of ice-induced, high- or low-flow levels on safety-related facilities and water supplies; (3) potential effects of a surface ice sheet to reduce the volume of available liquid water in safety-related water reservoirs; (4) potential effects of ice in producing forces on, or causing blockage of, safety-related facilities; (5) potential effects of seismic and non-seismic data on the postulated worst-case icing scenario for the proposed plant site; (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.7.2 *Summary of Application*

This section of the COL FSAR addresses the site-specific information about ice effects. The applicant addressed the information as follows:

AP1000 COL Information Item

- LNP COL 2.4-2

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 19 of the AP 1000 DCD.

> Combined License applicants referencing the AP1000 design will address the following site-specific information on historical flooding and potential flooding factors, including the effects of local intense precipitation:
>
> - Probable Maximum Flood on Streams and Rivers – Site-specific information that will be used to determine design basis flooding at the site. This information will include the probable maximum flood on streams and rivers.
> - Dam Failures – Site-specific information on potential dam failures.
> - Probable Maximum Surge and Seiche Flooding – Site-specific information on probable maximum surge and seiche flooding.

- Probable Maximum Tsunami Loading – Site-specific information on probable maximum tsunami loading.
- Flood Protection Requirements – Site-specific information on flood protection requirements or verification that flood protection is not required to meet the site parameter of flood level.

No further action is required for sites within the bounds of the site parameter for flood level.

2.4.7.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification and evaluation of ice effects, and the associated acceptance criteria, are described in Section 2.4.7 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying ice effects are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The related acceptance criteria are provided in the following RGs:

- RG 1.59, "Design Basis Floods for Nuclear Power Plants" (NRC 1977a), as supplemented by best current practices

- RG 1.102, "Flood Protection for Nuclear Power Plants" (NRC 1976b).

2.4.7.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.7 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to site-specific ice effects. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

2.4.7.4.1 Ice Conditions and Historical Ice Formation

Information Submitted by the Applicant

The applicant reviewed the historical temperature records from the NWS Cooperative Observer Station in Ocala, Florida. The monthly average minimum temperatures for the months of December, January, and February for the period 1971–2000 were 8.5, 7.6 and 8.3 ºC (47.3, 45.7, and 47 ºF), and the corresponding monthly mean temperatures were 15.3, 14.5, and 15.5 ºC (59.5, 58.1, and 59.9 ºF). The applicant concluded that ice formation on large bodies of water in the vicinity of the LNP site is unlikely and would not be severe enough to adversely affect the operation of safety-related SSCs.

NRC Staff's Technical Evaluation

The staff reviewed air temperature data from NOAA Cooperative Stations near the LNP site to evaluate the possibility of ice formation in the vicinity of the LNP site. The staff found several first-order stations located near the LNP site as listed in Table 2.4.7-1.

Table 2.4.7-1. First-Order NOAA NWS Cooperative Stations Located near the LNP Site

Name	County	Start Date	End Date
Inglis 3E	Levy	August 1, 1948	September 30, 1951
Morriston	Levy	March 1, 1940	February 28, 1942
Rockwell	Marion	August 1, 1899	June 30, 1919
Inverness 3 SE	Citrus	February 1, 1899	April 30, 2010
Ocala	Marion	January 1, 1892	February 28, 2010
Ocala 2NE	Marion	January 1, 1946	January 31, 1966

Of the stations near the LNP site, only those at Ocala and Inverness have long-term and current observations. The staff used these two meteorological stations to estimate characteristics of air temperature near the LNP site (Table 2.4.7-2).

Table 2.4.7-2. Statistics of Low Air Temperatures near the LNP Site

Statistics	Inverness	Ocala
Lowest daily mean air temperature	-4.4 ºC (24 ºF) on 2/14/1899	-3.6 ºC (25.5 ºF) on 12/24/1989
Number of days with daily mean air temperature below freezing	14 of 31,983	19 of 40,189
Longest period with daily mean air temperature at or below 0 ºC (32 ºF)	2 (three times)	2 (twice)
Longest period with daily mean air temperature at or below -7.8 ºC (18 ºF)	none	none

The staff independently determined that mean daily air temperature rarely (once in 2000 days) falls below freezing at the Inverness and Ocala stations. The longest duration over which mean daily air temperature was at or below freezing was 2 days at both Inverness and Ocala stations. There were no periods when mean daily air temperature fell below -7.8 ºC (18 ºF). Frazil ice forms in turbulent, supercooled water that is not covered by an ice layer but is directly in contact with the atmosphere with air temperature below -7.8 ºC (18 ºF) (USACE 2002). The staff concluded that ice formation, including frazil formation near the LNP site, is an unlikely event.

The LNP sites would host AP1000 units, which do not rely on an external safety-related source of water for safe shutdown. Therefore, the staff concluded that ice formation at the LNP site would not adversely affect safety-related SSCs for Units 1 and 2.

2.4.7.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.7.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed site characteristics and other hydrometeorological parameters related to ice formation at or near the plant site, and that there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.7 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses COL Information Item 2.4-2.

2.4.8 Cooling-Water Canals and Reservoirs

2.4.8.1 *Introduction*

FSAR Section 2.4.8 addresses the cooling-water canals and reservoirs used to transport and impound water supplied to the safety-related SSCs. Section 2.4.8 of this SER presents an evaluation of the following topics to verify their hydraulic design basis: (1) design bases postulated and used by the applicant to protect structures such as riprap, inasmuch as they apply to safety-related water supply; (2) design bases of canals pertaining to capacity, protection against wind waves, erosion, sedimentation, and freeboard and the ability to withstand a PMF (surges, etc.), inasmuch as they apply to a safety-related water supply; (3) design bases of reservoirs pertaining to capacity, PMF design basis, wind-wave and run-up protection, discharge facilities (e.g., low-level outlet, spillways, etc.), outlet protection, freeboard, and erosion and sedimentation processes inasmuch as they apply to a safety-related water supply; (4) potential effects of seismic and non-seismic information about the postulated hydraulic design bases of canals and reservoirs for the proposed plant site; and (5) any additional information requirements prescribed in the “Contents of Application” sections of the applicable subparts to 10 CFR Part 52.

2.4.8.2 *Summary of Application*

Safety systems for the AP1000 are designed to function without safety-related support systems such as component cooling water and service water. None of the safety-related equipment requires cooling water to affect a safe shutdown or mitigate the effects of design basis events. Heat transfer to the ultimate heat sink is accomplished by heat transfer through the containment shell to air and water flowing on the outside of the shell supplied by a passive containment cooling water tank. Therefore, the AP1000 design does not rely on service water and component cooling water systems to provide safety-related safe shutdown. There are no COL items related to cooling-water canals and reservoirs.

2.4.8.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification of design considerations for cooling-water canals and reservoirs, and the associated acceptance criteria, are described in Section 2.4.8 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for cooling-water canals and reservoirs are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The related acceptance criteria are provided in the following RGs:

- RG 1.59, “Design Basis Floods for Nuclear Power Plants” (NRC 1977a), as supplemented by best current practices

- RG 1.102, “Flood Protection for Nuclear Power Plants” (NRC 1976b).

2.4.8.4 *Technical Evaluation*

<u>*Information Submitted by the Applicant*</u>

The applicant stated that safety systems of the AP1000 reactor are designed to function without safety-related support systems such as component cooling water and service water. Heat transfer to the ultimate heat sink (UHS) occurs through the containment shell to the atmosphere and water supplied from a passive containment cooling-water tank. The applicant concluded, therefore, that no design bases for cooling-water canals or reservoirs are needed.

NRC Staff's Technical Evaluation

The staff reviewed the function of the AP1000 UHS and concluded that no external source of safety-related water is needed apart from the initial filling and occasional makeup water to the passive containment cooling-water storage tank located above the containment vessel and the passive containment cooling ancillary water storage tank located at ground level near the auxiliary building. Therefore, no safety-related cooling-water canals or reservoirs are needed at the LNP site with a permanent external source of water supply.

2.4.8.4.1 Cooling-Water Canals

Information Submitted by the Applicant

The applicant stated that safety systems of the AP1000 reactor are designed to function without safety-related support systems such as component cooling water and service water. Heat transfer to the UHS occurs through the containment shell to the atmosphere and water supplied from a passive containment cooling-water tank. The applicant concluded, therefore, that no design bases for cooling-water canals or reservoirs are needed.

NRC Staff's Technical Evaluation

The staff reviewed the function of the AP1000 UHS and concluded that no external source of safety-related water is needed apart from the initial filling and occasional makeup water to the passive containment cooling-water storage tank located above the containment vessel and the passive containment cooling ancillary water storage tank located at ground level near the auxiliary building. Therefore, no safety-related cooling-water canals or reservoirs are needed at the LNP site with a permanent external source of water supply.

2.4.8.4.2 Reservoirs

Information Submitted by the Applicant
The applicant stated that safety systems of the AP1000 reactor are designed to function without safety-related support systems such as component cooling-water and service water. Heat transfer to the UHS occurs through the containment shell to the atmosphere and water supplied from a passive containment cooling-water tank. The applicant concluded, therefore, that no design bases for cooling-water canals or reservoirs are needed.

NRC Staff's Technical Evaluation
The staff reviewed the function of the AP1000 UHS and concluded that no external source of safety-related water is needed apart from the initial filling and occasional makeup water to the passive containment cooling-water storage tank located above the containment vessel and the passive containment cooling ancillary water storage tank located at ground level near the

auxiliary building. Therefore, no safety-related cooling-water canals or reservoirs are needed at the LNP site with a permanent external source of water supply.

2.4.8.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.8.6 *Conclusion*

The staff reviewed the application and confirmed that the scope of Section 2.4.8 is not relevant to the LNP COL.

2.4.9 Channel Diversions

2.4.9.1 *Introduction*

LNP FSAR Section 2.4.9 addresses channel diversions. It evaluates plant and essential water supplies used to transport and impound water supplies to ensure that they will not be adversely affected by stream or channel diversions. The evaluation includes stream channel diversions away from the site (which may lead to a loss of safety-related water) and stream channel diversions toward the site (which may lead to flooding). In addition, in such an event, it must be ensured that alternate water supplies are available to safety-related equipment.

Section 2.4.9 of this SER presents an evaluation of the following specific areas: (1) historical channel migration phenomena including cutoffs, subsidence, and uplift; (2) regional topographic evidence that suggests a future channel diversion may or may not occur (used in conjunction with evidence of historical diversions); (3) thermal causes of channel diversion, such as ice jams, which may result from downstream ice blockages that may lead to flooding from backwater or upstream ice blockages that can divert the flow of water away from the intake; (4) potential for forces on safety-related facilities or the blockage of water supplies resulting from channel migration-induced flooding (flooding not addressed by hydrometeorological-induced flooding scenarios in other sections); (5) potential of channel diversion from human-induced causes (i.e., land-use changes, diking, channelization, armoring, or failure of structures); (6) alternate water sources and operating procedures; (7) potential effects of seismic and non-seismic information about the postulated worst-case channel diversion scenario for the proposed plant site; (8) any additional information requirement prescribed in the “Contents of Application” sections of the applicable subparts to 10 CFR Part 52.

2.4.9.2 *Summary of Application*

Safety systems for the AP1000 are designed to function without safety-related support systems such as component cooling water and service water. None of the safety-related equipment requires cooling water to affect a safe shutdown or mitigate the effects of design basis events. Heat transfer to the ultimate heat sink is accomplished by heat transfer through the containment shell to air and water flowing on the outside of the shell supplied by a passive containment cooling water tank. Therefore, the AP1000 design does not rely on service water and component cooling water systems to provide safety-related safe shutdown. There are no COL items related to cooling-water canals and reservoirs. There are no COL items related to channel diversions.

2.4.9.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification and evaluation of channel diversions, and the associated acceptance criteria, are described in Section 2.4.9 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying and evaluating channel diversions are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The related acceptance criteria are provided in the following RGs:

- RG 1.59, “Design Basis Floods for Nuclear Power Plants” (NRC 1977a), as supplemented by best current practices

- RG 1.102, “Flood Protection for Nuclear Power Plants” (NRC 1976b).

2.4.9.4 *Technical Evaluation*

Information Submitted by the Applicant

The applicant stated that the CFBC is a man-made drainage structure that is not susceptible to migration or cutoff. The applicant concluded, based on gauge height data at two stations that no channel diversion of significance has occurred in approximately 35 years of record. The applicant concluded, based on the size of the Gulf of Mexico, that complete diversion of the Gulf is unlikely. The applicant stated, based on topographic characteristics, geological features, and low seismic activity in the drainage basin, that there is no possibility of a landslide-induced blockage that might limit flow of water into the CFBC from the Gulf of Mexico or from Lake Rousseau. The applicant also stated that because ice effects in the vicinity of the LNP site are considered unlikely, ice-induced diversion during winter months is also unlikely. The applicant stated that a potential for anthropogenic diversion of CFBC exists; however, because it is located in a relatively unpopulated area, the potential for such an event is unlikely.

The applicant stated that the AP1000 design does not have a safety-related cooling-water system and therefore, does not rely on service water and component cooling-water systems for safe shutdown.

NRC Staff's Technical Evaluation

The staff reviewed the function of the AP1000 UHS and concluded that no external source of safety-related water is needed apart from the initial filling and occasional makeup water to the passive containment cooling-water storage tank located above the containment vessel and the passive containment cooling ancillary water storage tank located at ground level near the auxiliary building. Therefore, the LNP units will not rely on any external source of water for safety-related use. The NRC staff concluded that any potential channel migration in the vicinity of the site would not affect safe shutdown of the plant.

The staff evaluated the possibility of a channel diversion-induced flood near the LNP site. The staff determined that the safety-related SSCs of the LNP units would be located in the Waccasassa River Basin, specifically in the Spring Run and Thousandmile Creek-Halverson Creek subbasins. Surface drainages in both of these subbasins drain directly to the Gulf, so they do not contribute flow to the Waccasassa River. The safety-related SSCs of the LNP units would be located near the upper portion of these two subbasins, where there are no named streams or watercourses and overland flow during large precipitation events is drained toward the west and southwest. Based on this review of topography and hydrology in the vicinity of the LNP site, the NRC staff determined that a future channel diversion is unlikely in the vicinity of the LNP site. The staff concluded therefore that the safety-related SSCs of the LNP units would be safe from adverse effects of any potential channel diversion.

The staff reviewed Section 2.4.9 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information related to this review topic. Because the AP1000 reactor design does not require makeup water from offsite for safety-related purposes, the staff determined that the scope of FSAR 2.4.9 is not relevant for the LNP COL.

2.4.9.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.9.6 *Conclusion*

The staff reviewed the application and confirmed that the scope of Section 2.4.9 is not relevant to the LNP COL.

2.4.10 Flooding-Protection Requirements

2.4.10.1 *Introduction*

FSAR Section 2.4.10 addresses the locations and elevations of safety-related facilities and those of structures and components required for protection of safety-related facilities. These requirements are then compared with design basis flood conditions to determine whether flood effects need to be considered in the plant's design or in emergency procedures.

Section 2.4.10 of this SER presents an evaluation of the following specific areas:
(1) safety-related facilities exposed to flooding; (2) type of flood protection (e.g., "hardened facilities," sandbags, flood doors, bulkheads, etc.) provided to the SSCs exposed to floods; (3) emergency procedures needed to implement flood protection activities and warning times available for their implementation reviewed by the organization responsible for reviewing issues related to plant emergency procedures; (4) potential effects of seismic and non-seismic information about the postulated flood protection for the proposed plant site; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.10.2 *Summary of Application*

This section of the COL FSAR addresses the needs for site-specific information about flood protection requirements. The applicant addressed the information as follows:

COL Information Items

- LNP COL 2.4-2

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 19 of the AP1000 DCD.

> Combined License applicants referencing the AP1000 design will address the following site-specific information on historical flooding and potential flooding factors, including the effects of local intense precipitation.
>
> - Probable Maximum Flood on Streams and Rivers – Site-specific information that will be used to determine design basis flooding at the site. This information will include the probable maximum flood on streams and rivers.
> - Dam Failures – Site-specific information on potential dam failures.
> - Probable Maximum Surge and Seiche Flooding – Site-specific information on probable maximum surge and seiche flooding.
> - Probable Maximum Tsunami Loading – Site-specific information on probable maximum tsunami loading.

- Flood Protection Requirements – Site-specific information on flood protection requirements or verification that flood protection is not required to meet the site parameter of flood level.

No further action if required for sites within the bounds of the site parameter for flood level.

This section of the SER relates to historical flooding and local intense precipitation.

- LNP COL 2.4-6

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.6 of Revision 19 of the DCD.

2.4.10.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification and evaluation of flooding protection requirements, and the associated acceptance criteria, are described in Section 2.4.10 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying and evaluating flooding protection requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

The related acceptance criteria are provided in the following RGs:

- RG 1.59, “Design Basis Floods for Nuclear Power Plants” (NRC 1977a), as supplemented by best current practices

- RG 1.102, “Flood Protection for Nuclear Power Plants” (NRC 1976b).

2.4.10.4 *Technical Evaluation*

Information Submitted by the Applicant

The applicant stated that the AP1000 site parameters bound the LNP site flood levels.

NRC Staff's Technical Evaluation

The NRC staff reviewed the applicant's FSAR and related RAI responses to determine that the maximum floodwater surface elevation at the LNP site is 15.17 m (49.78 ft) NAVD88. This results from a probable maximum storm surge combined with wind-induced setup, as described in Section 2.4.2 of this SER. The maximum floodwater surface elevation is below the nominal plant grade floor elevation of 15.5 m (51 ft) NAVD88. The staff concluded therefore, that the DCD maximum flood level parameter would not be exceeded. Therefore, no flood protection is required for LNP Units 1 and 2.

2.4.10.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.10.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed the information to demonstrate that the characteristics of the site fall within the site parameters specified in the DC rule, and that there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated information relative to the flood protection measures important to the design and siting of LNP Units 1 and 2. The staff finds that the applicant has considered the appropriate site phenomena in establishing the flood protection measures for SSCs. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.10 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site.

2.4.11 Low-Water Considerations

2.4.11.1 *Introduction*

FSAR Section 2.4.11 addresses natural events that may reduce or limit the available safety-related cooling-water supply. The applicant ensures that an adequate water supply will exist to shut down the plant under conditions requiring safety-related cooling.

Section 2.4.11 of this SER presents an evaluation of the following specific areas: (1) low water conditions due to the worst drought considered reasonably possible in the region; (2) effects of low water surface elevations caused by various hydrometeorological events and a potential blockage of intakes by sediment, debris, littoral drift, and ice because they can affect the safety-related water supply; (3) effects of low water on the intake structure and pump design bases in relation to the events described in SAR Sections 2.4.7, 2.4.8, 2.4.9, and 2.4.11, which consider the range of water supply required by the plant (including minimum operating and shutdown flows during anticipated operational occurrences and emergency conditions) compared with availability (considering the capability of the UHS to provide adequate cooling water under conditions requiring safety-related cooling); (4) use limitations imposed or under discussion by Federal, State, or local agencies authorizing the use of the water; (5) potential effects of seismic and non-seismic information about the postulated worst-case low water scenario for the proposed plant site; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.11.2 *Summary of Application*

This section of the COL FSAR addresses the impacts of low water on water supply. The applicant addressed the information as follows:

AP1000 COL Information Item

- LNP COL 2.4-3

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.3 of Revision 19 of the AP1000 DCD.

> Combined License applicants will address the water supply sources to provide makeup water to the service water system cooling tower.

2.4.11.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the low water considerations, and the associated acceptance criteria, are described in Section 2.4.11 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for identifying the effects of low water are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.
- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the

limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.11.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.11 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to the low water considerations. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

Information Submitted by the Applicant regarding Low Flow in Rivers and Streams

The applicant provided an analysis of low flow in the Withlacoochee River using observed data at five USGS streamflow gauging stations.

Information Submitted by the Applicant regarding Historical Low Water

The applicant provided an analysis of low flow in the Withlacoochee River using observed data at five USGS streamflow gauging stations. The applicant compared the dates of the lowest observed water levels with those of hurricane occurrences but did not find any relationship between the two. The applicant concluded that low flow events are more likely to be caused by other effects, such as droughts.

Information Submitted by the Applicant regarding Heat Sink Dependability Requirements

The applicant stated that the UHS for the AP1000 design would not be affected by any low flow events because it does not rely on service water and component cooling-water systems. Water withdrawn from the CFBC would only be used to provide normal operational needs.

NRC Staff's Technical Evaluation

The staff reviewed the AP1000 DCD to evaluate the impact of low water conditions in the vicinity of the LNP site on the safety of the LNP units. Since no external water source is required for safe emergency shutdown, the staff determined that low water conditions would have no impact on the safety of the LNP units. There are no site characteristics in the DCD associated with low water conditions.

2.4.11.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.11.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed the required information, that there are no site characteristics in the DCD associated with low water conditions, and that there is no outstanding information required to be addressed in the COL FSAR related to this section..

As set forth above, the applicant has presented and substantiated information related to the low water effects important to the design and siting of this plant. The staff finds that the applicant has considered the appropriate site phenomena in establishing the design bases for SSCs. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.11 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses COL information item 2.4-3.

2.4.12 *Groundwater*

2.4.12.1 *Introduction*

FSAR Rev. 4 Section 2.4.12 describes the hydrogeological characteristics of the site. The most significant objective of groundwater investigations and monitoring at this site is to evaluate the effects of groundwater on plant foundations. The evaluation is performed to ensure that the maximum groundwater elevation remains below the DCD site parameter value. The other significant objectives are to examine whether groundwater provides any safety-related water supply; to determine whether dewatering systems are required to maintain groundwater elevation below the required level; to measure characteristics and properties of the site needed to develop a conceptual site model of groundwater movement; and to estimate the direction and velocity of movement of potential radioactive contaminants.

This section presents an evaluation of the following specific areas: (1) identification of the aquifers, types of onsite groundwater use, sources of recharge, present withdrawals and known and likely future withdrawals, flow rates, travel time, gradients and other properties that affect the movement of accidental contaminants in groundwater, groundwater levels beneath the site, seasonal and climatic fluctuations, monitoring and protection requirements, and manmade changes that have the potential to cause long-term changes in local groundwater regime; (2) effects of groundwater levels and other hydrodynamic effects of groundwater on the design bases of plant foundations and other SSCs important to safety; (3) reliability of groundwater resources and related systems used to supply safety-related water to the plant; (4) reliability of dewatering systems to maintain groundwater conditions within the plant's design bases; (5) potential effects of seismic and non-seismic information on the postulated worst-case groundwater conditions for the proposed plant site; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.12.2 *Summary of Application*

This section of the COL FSAR addresses groundwater conditions in terms of impacts on structures and water supply. The applicant addressed these issues as follows:

AP1000 COL Information Item

- LNP COL 2.4-4

This COL item is addressed by FSAR Section 2.4.12. In particular, this section addresses the site-related parameter for groundwater level that is specified in Table 2-1 of Revision 19 of the DCD, and is defined and discussed in Section 2.4.1.4 of Revision 19 of the DCD.
Section 2.4.1.4 states:

> Combined License applicants referencing the AP1000 certified design will address site-specific information on groundwater. No further action is required for the sites within the bounds of the site parameter for groundwater.

2.4.12.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for groundwater, and the associated acceptance criteria, are described in Section 2.4.12 of NUREG-0800.

The applicable regulatory requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.12.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.12 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to groundwater. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

2.4.12.4.1 Hydrogeological Description and Onsite Use of Groundwater

Information Submitted by the Applicant

The applicant stated that the LNP site is located on the Floridan platform, which consists of a sequence of Mesozoic and Cenozoic age shallow marine carbonate and evaporite sediments approximately 5,000 m (16,000 ft) thick. The site is located in the Gulf Coastal Lowlands, a subdivision of Florida's mid-peninsular physiographic zone. Much of the Gulf Coastal Lowlands has karst topography, an irregular terrain caused when near-surface carbonate rocks are dissolved by infiltrating rainwater.

The applicant described aquifers at the LNP site as consisting of a surficial aquifer, composed of unconsolidated Quaternary age sediments, and the deeper Floridan aquifer system found in the deeper predominately carbonate rocks of Miocene to Paleocene age. The Floridan aquifer system is extensive and receives recharge from a large area extending into Georgia, Alabama, and South Carolina. The Floridan aquifer system in Florida ranges in thickness from about 150 m (500 ft) to over 550 m (1800 ft) and consists of the Upper and Lower Floridan aquifers. The Upper Floridan and Lower Floridan aquifers are separated by low-permeability evaporite deposits and dense dolostones that form the middle confining unit (MCU). The MCU can be up to 122 m (400 ft) thick in the vicinity of the LNP site.

The Upper Floridan aquifer was described as the main source of potable water and spring flow in west-central Florida. The underlying Lower Floridan aquifer contains saline water and is not used as a potable water source near the LNP site. Site investigation boreholes drilled to as much as 152 m (500 ft) bgs (below ground surface) did not encounter the MCU (the bottom of the Upper Floridan aquifer) because it is below this depth.

The applicant described the local surficial aquifer as composed of sands. The applicant described the surficial aquifer as being recharged by wetlands mainly associated with cypress tree growth areas. The surficial aquifer in turn provides substantial recharge to the underlying Floridan aquifer system. Sands of the surficial aquifer grade into the carbonate-derived silty sediments at the top of the underlying Avon Park Formation (the uppermost geological formation within the Floridan aquifer that is present locally). The applicant stated that the thickness of the surficial aquifer at the LNP site varies from less than 3 m (10 ft) to about 60 m (200 ft) and the average thickness is approximately 15 m (50 ft). The applicant further described the surficial aquifer as being hydraulically connected to the Floridan aquifer. The water table in the surficial aquifer was generally found at depths of less than 1.5 m (5 ft). The water table varies seasonally depending on the amount of rainfall.

The applicant stated that the Upper Floridan aquifer is highly productive with transmissivity (thickness multiplied by hydraulic conductivity) estimated to range from approximately 4,645 to 9,290 m^2/d (50,000 to 100,000 ft^2/d) in the vicinity of the LNP site.

The reported site investigations included the drilling of geotechnical borings; installation and monitoring of wells completed in the surficial and upper bedrock aquifers; performance of slug tests and pumping tests; and analysis of water and soil samples. The applicant stated that there is no current onsite use of groundwater at the LNP site. The applicant indicated that general plant water supply for the new units, including service water tower drift and evaporation, potable water supply, raw water to the demineralizer, fire protection, and media filter backwash, will be provided by water supply wells completed in the Upper Floridan aquifer. The average flow rate needed was predicted to be 3,337 L/min (881.5 gpm).

NRC Staff's Technical Evaluation

The staff reviewed the information provided by the applicant in the FSAR regarding regional and site hydrogeology, groundwater conditions, and onsite groundwater use. The staff found the applicant's regional information to be comparable to the description provided in the "Ground Water Atlas of the United States" (USGS 1990) and in reports published by the Florida Geological Survey (Rupert 1988; Arthur et al. 2001). The staff confirmed that freshwater aquifers at the site include the uppermost surficial aquifer and the thicker and more extensive Upper Floridan aquifer. The staff also confirmed that no confining unit exists between the surficial and Upper Floridan aquifer systems in this area, and that these two aquifers are hydraulically connected. The staff found that hydraulic conductivity of the surficial aquifer is generally lower than that of the Upper Floridan aquifer. However, karst features that may be associated with some of the wetlands on the LNP site could result in areas of enhanced vertical hydraulic conductivity and connection between the surface and the Upper Floridan aquifer (White 1988). Neither of the aquifers is classified as a sole-source aquifer. The closest sole-source aquifer is the Volusia Sole-Source Aquifer, located approximately 80 mi east of the LNP site (EPA 2011).

The staff issued RAI 2.4.12-01 requesting additional information about groundwater chemistry as it relates to the transport properties of the subsurface. In response, the applicant provided groundwater chemistry data from the site monitoring wells and information related to the effects of groundwater chemistry on the transport of potential radioactive contaminants (ML092150960). The staff reviewed the information and determined that the information was adequate to support the analysis of transport from a hypothetical spill to groundwater presented in Section 2.4.13 of this report.

The staff found that there is no current onsite use of groundwater at the LNP site. Fresh groundwater from the Upper Floridan aquifer would be used for general plant water supply at LNP Units 1 and 2, but not for reactor cooling water. Groundwater will be withdrawn at an average of 4,153 L/min (1,097 gpm, or 1.58 mgd) to provide makeup water for service water tower drift and evaporation, potable water supply, raw water to the demineralizer, fire protection, and media filter backwash. The staff determined that the groundwater supply's lack of safety function is consistent with the uses stated for groundwater, and with provisions for safety-related water supply from other sources, as described in the FSAR Revision 2.

2.4.12.4.2 Groundwater Sources, Present and Future Groundwater Use

Information Submitted by the Applicant

The applicant determined that within 40.2 km (25 mi) of the LNP site, the SWFWMD has issued approximately 53,670 well permits, and the Suwanee River Water Management District (SRWMD) has issued 918 well permits. The applicant also determined that there are 268 public water supply systems within a 40.2 km (25-mi) radius of the LNP site. Of these, 46 public water supply systems serving 10,300 customers and having total design capacity of approximately 25 MLd (6.6 Mgd) are within 16 km (10 mi) of the LNP site. A total of 64 wells draw water from the Upper Floridan aquifer for these 46 public water supply systems. The applicant also found that three municipal/city systems account for approximately 7.2 MLd (1.9 Mgd), or 30 percent of the total public water supply design capacity within 16.1 km (10 mi) of the LNP site. The numbers and types of permitted wells were tabulated by Township Range and Section in FSAR Revision 4. Information about public water supply wells was also presented in the FSAR.

The applicant indicated that SWFWMD projected an increase in water demand within Levy County from approximately 49.6 MLd (13.1 Mgd) in 1994 to approximately 68.5 MLd (18.1 Mgd) in 2020, an increase of 18.9 MLd (5.0 Mgd) or 38 percent (SWFWMD 1997). However, the applicant also found that water use actually decreased in Levy County between 1994 and 2005, when it was reported as approximately 35.9 MLd (9.5 Mgd).

The applicant conducted a land-use survey covering the area within 8 km (5 mi) of the LNP site to identify the nearest residents and collect information including the number and use of wells. The results showed that all of the residents within this area use groundwater to supply their potable water needs, and that the depths of these private water wells range from 6 m (20 ft) to 137 m (450 ft) bgs. The nearest residential well was found to be about 2.6 km (1.6 mi) northwest of the LNP site.

NRC Staff's Technical Evaluation

The staff reviewed the information provided in FSAR Rev. 4 on current groundwater use and checked the provided data through queries of electronic databases available from the SWFWMD (2011) and SRWMD. The staff found that information provided in the FSAR was accurate, but, as noted by the applicant, some wells in the database may no longer be in use. This would result in an over-estimate of groundwater users. The staff issued RAI 2.4.12-03 to request an explanation for why, as shown in FSAR Figures 2.4.12-206 to 2.4.12-210, the density of wells in the SWFWMD was apparently much greater than in the SRWMD. In response, the applicant indicated that the SWFWMD requires registration of all wells, including domestic wells, but the SRWMD does not require registration of domestic wells.

The staff found that information provided in the FSAR was accurate, but, as noted by the applicant, some wells in the database may no longer be in use. This would result in an over-estimate of groundwater users. The staff checked the documents (SWFWMD 1997; SWFWMD 2009b) cited in the FSAR and verified information presented regarding future water use. There is uncertainty in the projections of groundwater use because previously published projections indicate steadily increasing population and water use. However, groundwater use in the area has decreased since 1994. The staff determined that the projected future water use provided in LNP FSAR Rev 2 of approximately 68.5 MLd (18.1 Mgd) in the year 2020 is conservatively higher than the likely actual future use. Projected water use in the SRWMD through 2030 was presented in a Water Supply Assessment (SRWMD 2010b). The purpose of the assessment was to determine whether water supplies in the district will satisfy water demands for all uses in the 2010 to 2030 planning period while protecting the environment. The SRWMD assessment estimated a range of 17 to 45 percent increase in demand for public water supply over the 20-year period. The applicant's estimation of projected increase in groundwater use through 2020 is within this range.

The staff issued RAI 2.4.12-02 requesting additional information about the planned plant water supply wells, including the design of the wellfield and the projected impacts of pumping on transport pathways, surrounding surface waters, and adjacent offsite groundwater users. In response, the applicant provided details on the plant water supply wells, including location, number of wells, and peak and average expected flow rates (ML092150960). The applicant also referred to the results of a site groundwater model (ML092240668). However, this model was subsequently revised by the applicant based on staff's environmental RAI 5.2.2-4 (ML093620182) related to the LNP environmental impact statement. The new revision of the groundwater model was documented by the applicant (ML093620211). The staff reviewed the revised groundwater model (ML093620211) and found that it did achieve the goals of matching groundwater levels measured on the LNP site and in four other wells measured in the area by the USGS. Results from the predictive model simulations showed that annual average LNP groundwater usage is relatively small compared to the overall model water balance. The LNP average operational usage of 5.98 MLd (1.58 Mgd) represents only 0.8 percent of the total water flux (787 MLd [208 Mgd]) through the model domain. At this withdrawal rate, the LNP wellfield is predicted to decrease the surficial and Upper Floridan aquifer discharge to surface waterbodies within the model domain by approximately 1.5 MLd (0.4 Mgd), or about 2 percent of the total simulated groundwater discharge to rivers and lakes.

Based on the information provided on the planned water supply wells, expected pumping rates, and the revised model calculation of water level impacts, the staff determined that pumping of the water supply wells will have little effect on offsite groundwater users or surface waterbodies. Significant problems have resulted from overuse of groundwater in upland northeastern portion of the SRWMD (SRWMD 2010a). However, the location of the LNP site in the lower portion of the drainage basin results in adequate recharge of the aquifer to meet demand.

The staff also determined that the planned groundwater supply for the proposed units does not have a safety function, so a loss of the groundwater supply will not compromise plant safety.

2.4.12.4.3 Groundwater Levels and Movement

Information Submitted by the Applicant

The applicant characterized the hydrogeology of the LNP site using groundwater observations, well tests, laboratory tests, and examination of site topography and geology.

The applicant described the observation well network installed to monitor water levels and determine hydraulic gradients and groundwater flow paths for the surficial and bedrock aquifers in the vicinity of the LNP site. Nested well sites with shallow, intermediate, and deep monitoring wells were installed and monitored to determine vertical gradients between the surficial and bedrock aquifers and variations over time.

The applicant installed a pumping test well and 23 observation and monitoring wells in 2007. The pumping test well and 15 of the observation and monitoring wells were screened within the silt and sand of the surficial aquifer directly above the bedrock interface at depths of 4 to 10.4 m (13 to 34 ft) bgs. Seven wells were installed at depths of 37.2 to 46.9 m (122 to 154 ft) in the limestone of the Upper Floridan aquifer and two wells were installed at an intermediate depth of 20.7 to 24.1 m (68 to 79 ft) within the limestone bedrock of the Upper Floridan aquifer. Water levels were measured in the wells in March, June, September, and December of 2007 to determine the configuration of the potentiometric surface in the immediate vicinity of the LNP site. The applicant found that the depth to groundwater was between 0 and 2.4 m (0 and 8 ft) with the shallowest groundwater levels occurring during the spring. The applicant determined that the groundwater is shallow and unconfined, and that groundwater conditions are influenced by the topography of the LNP site. They described the groundwater as flowing from a topographic high of approximately 18.3 m (60 ft) NGVD29 in the eastern portion of the site toward a topographic low of approximately 10.7 m (35ft) NGVD29 in the southwest portion of the site. In the center portion of the site, where the topography is relatively flat, the groundwater surface also becomes relatively flat. The applicant found that no significant differences were observed in groundwater flow direction or gradient during the quarterly measurements or between the surficial and bedrock aquifer.

The applicant installed pressure transducers in two wells screened in the surficial aquifer and collected groundwater elevation data every 12 hours for more than a year. These wells were located at the approximate center of the footprints for each of the two new units. The applicant found that maximum groundwater elevations were observed during March 2007 and March 2008 at both wells. They also found that groundwater elevations were more than 2.1 m (7 ft) below nominal plant grade elevation and more than 2.4 m (8 ft) below nominal plant floor elevation between March 2007 and March 2008.

The applicant calculated horizontal gradients of 0.0003 to 0.0007 between pairs of upgradient and downgradient monitoring wells based on March 2007 water level measurements. The applicant found slightly greater hydraulic heads within the surficial aquifer compared to the bedrock Floridan aquifer based on measurements at the six nested well sites. Measured vertical gradients in March 2007 for all sets of wells ranged from 0.0003 to 0.006 based on the vertical distance between the mid-point of the well screens. The two well pairs (MW-15S/MW-16D and MW-13S/MW14D) located within the footprint of LNP 1 and LNP 2 had slight downward vertical gradients with elevation head differences of 0.17 and 0.08 m (0.55 and 0.27 ft), respectively, in September 2007. The applicant found that the vertical gradients between the surficial and bedrock aquifers remained consistent for all nested well sets during each quarterly gauging event. However, groundwater levels in both aquifers were found to be higher in the spring and lower in the fall.

NRC Staff's Technical Evaluation

The staff issued RAI 2.4.12-05 requesting the site groundwater elevation monitoring data (including the monitoring locations) and the available historical seasonal groundwater elevations in the vicinity of the LNP site. In response, the applicant provided a map of site monitoring locations and also provided the measured groundwater elevation data for the onsite monitoring wells, including quarterly monitoring events and hourly measurements collected using pressure transducers (ML092190616). The applicant's response also included electronic links to other nearby water level records available from the USGS.

The staff issued RAI 2.4.12-06 requesting that the applicant clarify the description of groundwater discharge areas in the FSAR. The applicant's response referred to the response to RAI 2.4.12-08 discussed below (ML092150960).

In RAI 2.4.12-07, the staff asked the applicant to clarify "the significance of vertical hydraulic gradients in relation to the selection of the most conservative plausible conceptual model for transport of radioactive liquid effluents in the subsurface." The applicant responded with an explanation that the observed downward gradients between the surficial and bedrock aquifer indicate that effluents would migrate downward into the bedrock aquifer (Upper Floridan aquifer) and that this assumption is appropriately conservative because permitted water supply wells are only completed in the Upper Floridan aquifer and not in the surficial aquifer (ML092150960). The applicant response also indicated that seepage velocities in the Upper Floridan aquifer are greater than those in the surficial aquifer.

The staff issued RAI 2.4.12-08 asking the applicant to clarify the interpretation of vertical groundwater gradients. The applicant responded with a clarification regarding the USGS identification of the LNP area as a recharge/discharge boundary and discussion of the onsite nested-well monitoring results that indicate a generally small but variable downward gradient (ML092150960). The applicant revised the FSAR to include the following text: "Regionally, the USGS has identified the area where the LNP site is located as a recharge/discharge boundary of the Floridan aquifer as shown in Figure 2.4.12-226. Site-specific vertical gradients observed quarterly from early 2007 through early 2008 were all downward and low in magnitude, ranging from 0.0002 to 0.018 (FSAR Table 2.4.12-209)."

The staff reviewed the information provided regarding groundwater levels and the direction and gradient of groundwater movement. The staff determined that the applicant had adequately characterized groundwater movement under pre-construction site conditions through measurements of water levels in both the surficial aquifer and upper Floridan aquifer. Groundwater was found to flow predominately to the southwest with a maximum measured horizontal gradient of 0.0007. The measured vertical component of the pre-construction gradient was consistently downward with a maximum measured gradient of 0.018. The staff agrees that the vertical component of the gradient will continue to be downward during the operational period because pumping of the proposed water supply wells is likely to lower the hydraulic head in the Upper Floridan aquifer. The vertical gradient indicates that any accidentally released contaminants would migrate downward into the bedrock aquifer (Upper Floridan aquifer). However, the staff found that there is uncertainty in the applicant's estimate of future groundwater levels during the period of plant operations because of planned changes to the site, including the placement of fill, changes in surface cover, and installation of stormwater drainage ditches and ponds.

2.4.12.4.4 Site Hydrogeologic Characteristics

Information Submitted by the Applicant

The applicant conducted slug tests in 23 wells to estimate saturated hydraulic conductivity of the surficial and Upper Floridan aquifers. Results ranged from 0.27 m/d (0.9 ft/d) to 8.7 m/d (28.6 ft/d) for the surficial aquifer and from 0.73 m/d (2.4 ft/d) to 16.6 m/d (54.4 ft/d) for the Upper Floridan aquifer.

An aquifer pumping test was also performed at well PW-1. The initial pumping test analysis provided in FSAR Rev 2 resulted in transmissivity values (hydraulic conductivity multiplied by aquifer thickness) ranging from 121 m^2/d (1300 ft^2/d) to 204 m^2/d (2200 ft^2/d) and specific yield estimates from 0.012 to 0.17. The pumping test analysis was later revised and estimates of hydraulic conductivity and groundwater seepage velocity were revised in response to RAIs issued by the NRC staff.

NRC Staff's Technical Evaluation

The staff reviewed information provided in FSAR Rev 2 on site hydraulic characteristics and the related RAI responses. The staff reviewed the multi-layer transient analyses of the applicant's aquifer pumping test provided in response to RAIs 2.4.12-11 (ML092150960) and 2.4.12-22 (ML101740492) and determined that the analysis methods are valid for the test conditions and that these tests provide a reasonable estimate of site-specific hydraulic conductivity of 36.6 to 39.6 m/d (120 to 130 ft/d) for the Upper Floridan aquifer in the vicinity of the test wells. The Multi-Layer Unsteady (MLU) state model used in the analyses tended to over-predict pump-test-induced drawdown at some locations and under-predict drawdown at other locations. However, that is expected because of heterogeneity within the aquifers, and the scatter plots comparing the observed and simulated drawdown response for all monitoring wells indicated a reasonable composite match of the data.

The staff issued RAI 2.4.12-09 asking the applicant to clarify whether any spatial trend or regularities are evident in the hydraulic conductivities measured by the slug tests on the LNP site. The applicant responded by providing maps of the slug test results for both the surficial and bedrock aquifers and stated that values vary across the site by up to an order of magnitude, but do not appear to show any spatial trend (ML092150960). The NRC staff determined that, based on the maps provided, the response was sufficient to meet the requested information need. However, the results of the slug tests were found to not be sufficiently representative of site aquifer conditions. These concerns are addressed in RAI 2.4.12-10, 2.4.12-11 and 2.4.12-12 discussed below.

RAI 2.4.12-10 was issued asking the applicant to clarify the apparent discrepancy in the estimated transmissivity range presented in FSAR Revision 0, Section 2.4.12.1.1 and the average transmissivity values derived from slug tests and to discuss which of these values is most representative of actual site conditions. The applicant responded by explaining that the transmissivity values presented in FSAR Revision 0, Section 2.4.12.1.1 were regional estimates from literature sources and not site-specific.

RAI 2.4.12-11 requested that the applicant justify the approach adopted for analysis of pumping tests in the FSAR. The applicant responded by providing new analyses of the three aquifer pumping tests (ML092150960). The new analyses were based on a transient multi-layer analysis using the MLU model. The applicant used an iterative analysis approach because analysis of the Upper Floridan aquifer data required the properties of the surficial aquifer as input, and analysis of the surficial aquifer data required the properties of the Upper Floridan aquifer as input. The analysis resulted in a single set of hydraulic property values that best matched the observed response at all available monitoring locations, rather than fitting separate sets of hydraulic properties to different locations. The applicant summarized the results of the aquifer pumping tests and determined that transmissivity of the Upper Floridan aquifer at the site ranged from 5760 to 6410 m^2/d (62,000 to 69,000 ft^2/d), with an assumed Upper Floridan aquifer thickness of 158.5 m (520 ft). The applicant calculated an Upper Floridan aquifer hydraulic conductivity from the revised pumping test analyses of 36.6 to 39.6 m/d (120 to 130 ft/d) based on an aquifer thickness of 158.5 m (520 ft). The NRC staff reviewed the calculation package including the pumping test methods and analyses and determined that the analysis methods are valid for the test conditions and that these tests provide a reasonable estimate of site-specific hydraulic conductivity for the Upper Floridan aquifer in the vicinity of the test wells. The hydraulic conductivity may be higher in the upper part of the aquifer and lower in the deeper part based on observations of increasing amounts of evaporate and quartz-filled porosity below depths of 121.9 m (400 ft) noted in the response to RAI 2.4.12-10 (ML092150960).

The staff issued RAI 2.4.12-12 asking the applicant to discuss selection of hydraulic conductivity estimates used in the seepage velocity calculations and whether these result in conservative estimates of groundwater velocity. The applicant responded by describing that the hydraulic conductivity estimates of 8.72 and 16.6 m/d (28.6 and 54.4 ft/d) for the surficial and Upper Floridan aquifers, respectively, were considered conservative when used as a single value to characterize hydrogeological conditions for the entire LNP site because of regional and local variability of this property within the aquifers. As a follow-up to the applicant's response to RAI 2.4.12-12, the staff issued new RAI 2.4.12-22 asking the applicant to discuss how the seepage velocity reported in the FSAR based on a hydraulic conductivity of 16.6 m/d (54.4 ft/d) was conservative when higher hydraulic conductivity results were indicated by reanalysis of the aquifer pumping tests and the revised groundwater model (ML093620211). The applicant response described conservative assumptions in the FSAR Section 2.4.13 transport calculations including the receptor location on the property boundary and use of a 76-m (250-ft) aquifer thickness when the total Upper Floridan aquifer thickness is estimated at 158.5 m (520 ft). The applicant also referred to the slug test results ranging from 0.73 to 16.6 m/d (2.4 to 54.4 ft/d). The applicant provided a more detailed map of hydraulic conductivity estimated from calibration of the revised groundwater flow model (ML093620211) that showed transmissivity ranging between 736 and 2734 m^2/d (7,920 and 29,429 ft^2/d) between the proposed plants and the property boundary in the direction of groundwater flow. The applicant response continued to support use of a hydraulic conductivity value of 16.6 m/d (54.4 ft/d) in the seepage velocity calculations as being conservative based on regional and local variability within the aquifer. However, the applicant also provided an alternative seepage velocity calculation based on a hydraulic conductivity of 39.6 m/d (130 ft/d) and used this value for a "bounding analysis" of contaminant transport presented in the response to staff's RAI 02.04.13-13 (ML092150960).

The staff found that the hydraulic conductivity range provided by the applicant was not based on all available information. Instead, it was based only on the results of the slug tests and did not consider the new pumping test analyses provided in the response to RAI 2.4.12-10 or the results of the recalibrated version of the District Wide Regulation Model Version 2 (DWRM2) groundwater flow model (ML093620211). The range of hydraulic conductivity calculated by the applicant from the pumping tests was 36.6 to 39.6 m/d (120 to 130 ft/d) for the Upper Floridan aquifer compared to estimates of 8.72 and 16.6 m/d (28.6 and 54.4 ft/d) used in the seepage velocity calculations. The applicant's estimates of hydraulic conductivity were also low compared to the transmissivity (hydraulic conductivity multiplied by aquifer thickness) results of the recalibrated version of the DWRM2 groundwater flow model (ML093620211). The staff reviewed the follow-up RAI 2.4.12-22 requesting more information about the hydraulic conductivity estimates used in the seepage velocity calculations and determined that the hydraulic conductivity range of 36.6 to 39.6 m/d (120 to 130 ft/d) estimated from the aquifer pumping tests (ML092150960) is more representative of site conditions than the slug test results presented in LNP FSAR Revision 2, because the pumping test analysis accounts for vertical flow within and between the aquifers and because the pumping tests are affected by a much larger volume of rock within the aquifer than slug tests. The staff also found that the transmissivity values calculated from the MLU analysis of the aquifer pumping tests (ML092150960) for both the surficial and Upper Floridan aquifers fall within the ranges predicted by the revised groundwater model for the LNP site (ML093620211). The applicant revised the FSAR to include the results of the MLU aquifer test analyses.

The staff agreed with the applicant's assessment that the hydraulic conductivity may be higher in the upper part of the aquifer and lower in the deeper part of the aquifer. The staff agreed because increasing amounts of filled porosity below depths of 122 m (400 ft) were observed in samples from boreholes.

The staff issued RAI 2.4.12-14 asking the applicant to justify the use of the porous media concept for estimating seepage velocity and describe whether preferential flow paths associated with fracturing and solution cavities in carbonate rock aquifers at the LNP site should be considered when developing conservative estimates of groundwater velocity. The applicant responded by providing discussion and references concerning the use of a porous media conceptual model for flow and transport calculations in the Upper Floridan aquifer (ML092150960). The applicant included a reference to the EPA document (EPA 1989), which describes the Upper Floridan aquifer as having flow velocities that are likely to be slower than those found in "conduit-flow" aquifers. The applicant argued that the porous media concept assuming diffuse flow through interconnected pores was appropriate for developing a conservative estimate of groundwater flow velocity.

The staff reviewed the applicant's response to RAI 2.4.12-14 and determined that it would be appropriate to use a porous media conceptual model for the groundwater velocity (see page velocity) calculations if the effective porosity value used in the calculations represents the secondary porosity features (fractures and solution channels) of the groundwater flow system rather than the overall porosity of the system. The staff found that this, usually lower, secondary porosity is likely to control the first arrival of groundwater contaminants at a downgradient location within the Upper Floridan aquifer near the LNP site. However, the applicant's seepage velocity calculations presented in the LNP FSAR were based on an effective porosity estimate of 0.15 that pertains to the overall porosity of the limestone aquifer rather than the secondary fracture porosity. The applicant did not provide any site-specific measurements of effective porosity at the LNP site at the scale of the transport calculation. The staff found that published information indicates there is a possibility of preferential groundwater flow through fractures or solution cavities within the Upper Floridan aquifer in the vicinity of the LNP site (Knochenmus and Robinson 1996; Robinson 1995). According to a USGS report "Karst carbonate aquifers can be characterized by conduit flow along irregularly distributed, solution-enlarged fissures (channel porosity) in combination with diffuse flow through the more uniformly distributed, interconnected pores (rock porosity). The Floridan aquifer system of west-central Florida is in this category" (Knochenmus and Robinson 1996). Additional information from the "shallow" tracer test at the Old Tampa Well Field (Robinson 1996) demonstrates that secondary porosity features control the transport of dissolved contaminants in the Upper Floridan aquifer. The "shallow" tracer test was conducted in the upper 90 ft of the Upper Floridan aquifer over a distance of 61 m (200 ft) and resulted in an estimated effective porosity of 0.003 based on the early arrival of the tracer (Robinson 1996). The short travel time and low effective porosity was attributed to secondary aquifer porosity caused by fractures in the limestone.

Because of the lack of site-specific information about effective porosity at the scale of the contaminant transport scenario considered in Section 2.4.13, the staff issued an additional RAI 2.4.12-23 asking the applicant to provide additional discussion of how a porosity of 0.15 represented a conservative value or to justify the exclusion of in situ tests in the Upper Floridan aquifer that resulted in lower values of estimated effective porosity. The applicant responded by describing how the mean porosity value of 0.19 was calculated from porosity values compiled by the USGS for the Avon Park limestone formation (ML101740492). The applicant considered the lower porosity of 0.15 to be conservative, because it was smaller than the field-derived porosity of 0.19. The applicant also stated that, although lower values of porosity are found at some locations in the Upper Floridan aquifer, tests that produced these lower porosities were performed in the Suwannee and Ocala limestones, and these formations are more likely to have thin layers of higher conductivity rock compared to the Avon Park Formation. The applicant also described how tracer tests conducted over small distances are more likely to be dominated by flow through smaller-scale secondary porosity features but will tend to act more like an equivalent porous media over larger distances, as noted by Knochenmus and Robinson (1996). In addition, the applicant provided an alternative seepage velocity calculation based on an effective porosity of 0.05 and used this value for a "bounding analysis" of contaminant transport presented in the response to RAI 2.4.13-13 (ML101830016).

The staff reviewed the applicant's response to RAI 2.4.12-23 regarding effective porosity of the Upper Floridan aquifer (ML101740492). The staff agrees that the Avon Park limestone formation is more likely to behave as a continuous porous medium than the Suwannee or Ocala limestones. The staff also agrees that the longer travel distance of more than 1.6 km (1 mi) to an offsite groundwater user will increase the likelihood that the aquifer will behave as a continuous porous medium compared to tracer tests conducted over smaller distances. However, because of the lack of site-specific measurements of effective porosity and the difficulty of obtaining such estimates that would apply to the scale of the transport scenario, the staff does not concur that 0.15 is a conservative estimate with regard to the transport analysis. The staff concurs that the effective porosity of 0.05 proposed by the applicant as a more conservative alternative value, and used in an alternative seepage velocity calculation provided in the response to RAI 2.4.12-23, is a reasonably conservative parameter for the analysis of contaminant transport to an offsite groundwater user.

The applicant calculated seepage velocities and Darcy flux values between pairs of upgradient and downgradient monitoring wells. The applicant used the hydraulic gradient based on March 2007 water level measurements, the range of hydraulic conductivity values from the slug tests, and porosity values of 0.2 for the surficial aquifer and 0.15 for the Upper Floridan aquifer to calculate seepage velocity. The applicant determined porosity values based on four literature references. Resulting seepage velocities ranged from 0.0003 to 0.037 m/d (0.001 to 0.12 ft/d) for the surficial aquifer and 0.003 to 0.08 m/d (0.01 to 0.27 ft/d) for the Upper Floridan aquifer. The alternative seepage velocity calculation based on an effective porosity of 0.05 and hydraulic conductivity of 39.6 m/d (130 ft/d) used for the "bounding analysis" provided in RAI responses was 0.56 m/d (1.84 ft/d).

The staff reviewed calculated seepage velocities and Darcy flux values reported in FSAR Revisions 2. The use of measured gradients between pairs of monitoring wells based on March 2007 water level measurements were found to give a reasonable gradient. As discussed above, the staff does not concur that the hydraulic conductivity values from the slug tests or the porosity value of 0.15 for the Upper Floridan aquifer are conservative values in regard to the calculation of seepage velocity. The alternative seepage velocity calculation based on an effective porosity of 0.05 and hydraulic conductivity of 39.6 m/d (130 ft/d) used for the "bounding analysis" provided in RAI responses (ML101740492) was 0.56 m/d (1.84 ft/d) and the staff considers this to be a conservative value.

2.4.12.4.5 Effects of Groundwater Usage

Information Submitted by the Applicant

The applicant provided information about nondomestic groundwater use in the portion of Levy County that falls within the SWFWMD. Permitted nondomestic use in that area was stated to be 83.113 MLd (21.956 Mgd) in 2005. The applicant also described that only 29 MLd (7.677 Mgd) of that permitted amount was actually being used in 2005. Total groundwater demand in that area including non-permitted domestic use was 35.942 MLd (9.495 Mgd).

The average groundwater operational use by LNP was projected to be 4.8 MLd (1.269 Mgd) with a maximum use rate of 22.1 MLd (5.848 Mgd). The applicant stated that groundwater will also be withdrawn during temporary dewatering of site excavations and may be used for other purposes such as concrete mixing and dust control.

The applicant determined that the dewatering withdrawals and operational withdrawals of groundwater will not affect local groundwater users.

The applicant provided information about the plant water supply in an earlier section of LNP FSAR Revision 2.

NRC Staff's Technical Evaluation

The applicant's response to RAI 2.4.12-02 provided additional details of plant water supply wells including the design of the wellfield and the projected impacts of pumping on transport pathways, surrounding surface waters, and adjacent offsite groundwater users. The applicant provided the water supply well locations, number of wells, and peak and average expected flow rates (ML092150960).

The staff issued RAI 2.4.12-15 asking that the applicant "clarify the potential effects of groundwater pumping for plant water supply on groundwater levels, transport pathways, surface water, and other water users in the affected area." The applicant responded (ML092150960) by referring to the PEF source (ML092240668), which discussed MODFLOW modeling of groundwater levels, and responses to RAIs 2.4.12-02 (ML092150960) and 2.4.13-04 (ML092080078). However, the groundwater model described in the PEF source (ML092240668) was subsequently revised by the applicant as documented by PEF (ML093620211). The staff reviewed the results of the revised groundwater model as reported by PEF (ML093620211) and found that the applicant resolved RAI 2.4.12-15 by providing a defensible groundwater model that predicts the effects of pumping the water supply wells on the groundwater potentiometric surface. The staff found that the revised groundwater model achieved the goals of matching groundwater levels measured on the LNP site and in four other wells measured in the area by the USGS.

Results from the revised model simulations showed that annual average LNP groundwater usage is relatively small compared to the overall groundwater model water balance, that is, to the total amount of groundwater simulated to be flowing through the model. LNP average operational usage of 6 MLd (1.58 Mgd) represents only 0.8 percent of the total water flux (787 MLd [208 Mgd]) through the model domain. At the projected groundwater withdrawal rate, the LNP wellfield is predicted by the revised model to decrease the surficial and Upper Floridan aquifer discharge to surface waterbodies within the model domain by approximately 1.5 MLd (0.4 Mgd), or about 2 percent of the total groundwater discharge to rivers and lakes as simulated by the model.

The revised groundwater model showed that pumping of the water supply wells will have little effect on offsite groundwater users or surface waterbodies. The staff reviewed the applicant's response and determined, based on the information provided on the planned water supply wells, expected pumping rates, and the revised model calculation of water level impacts, that the response meets the requirements for this information need.

Although the staff did not independently run the applicant's model, the staff reviewed the model, including parameters used, boundary conditions, discretization, calibration results, and calculation validity, and on this basis determined that the results were adequate to estimate future impacts on groundwater use.

2.4.12.4.6 Subsurface Pathways

Information Submitted by the Applicant

In Section 2.4.12.3 of LNP FSAR Rev 4, the applicant refers to the previous Section 2.4.12.2, titled “Sources,” which discusses the locations of wells, and to Section 2.4.13.2, titled “Groundwater Scenarios,” concerning conservative analysis of critical groundwater pathways for a liquid effluent release at the site and the determination of groundwater and radionuclide travel times to the nearest downgradient groundwater user or surface waterbody.

In LNP FSAR Revision 2 Section 2.4.12.4.2, the applicant used water levels measured at onsite monitoring wells to determine flow directions and gradients. Seepage velocities and Darcy flux were calculated between pairs of upgradient and downgradient monitoring wells. Seepage velocity was calculated from the hydraulic gradient based on March 2007 water level measurements, the range of hydraulic conductivity values from the slug tests, and porosity values of 0.2 for the surficial aquifer and 0.15 for the Upper Floridan aquifer. The porosity values were determined based on four literature references. Resulting seepage velocities ranged from 0.0003 to 0.037 m/d (0.001 to 0.12 ft/d) for the surficial aquifer and 0.003 to 0.08 m/d (0.01 to 0.27 ft/d) for the Upper Floridan aquifer.

NRC Staff’s Technical Evaluation

The staff issued RAI 2.4.12-16 asking the applicant to describe plausible groundwater pathways for use in the analysis of transport of accidental liquid radioactive effluent release in the subsurface. The applicant responded by providing a discussion of the plausible potential groundwater pathways that were considered in the analysis of groundwater transport of radioactive releases to the subsurface (ML092150960). Pathways included in the RAI response considered transport to the surficial aquifer, transport from the surficial aquifer to the underlying Upper Floridan aquifer, transport through the Upper Floridan aquifer to nearby private and public wells, transport into the LNP retention pond and wetlands in the direction of groundwater movement, and transport to the Withlacoochee River. The applicant also considered the potential impact of the proposed LNP water supply wells on groundwater transport. Based on the revised groundwater model results (ML093620211), it was concluded that pumping of the supply wells could have a minor impact on groundwater transport. However, the pumping will not result in faster transport of contaminants to off-site users than under non-pumping conditions.

The staff reviewed the information provided in LNP FSAR Revision 2 and RAI responses concerning subsurface pathways for transport of radionuclides through groundwater and determined that all the plausible pathways had been considered. There are no other shallow aquifers that could provide a pathway for groundwater contaminants to move offsite and no other nearby surface water features that are considered potential receptors of groundwater contaminants.

2.4.12.4.7 Groundwater Monitoring or Safeguard Requirements

Information Submitted by the Applicant

The applicant described the monitoring programs that are planned to protect present and projected future groundwater users near the LNP site. The objectives of the groundwater monitoring programs were stated. Monitoring programs are planned for the pre-application period, construction, the preoperational period, and plant operation.

NRC Staff's Technical Evaluation

The staff issued RAI 2.4.12-17 asking the applicant to update FSAR Section 2.4.12.4 with a summary of the details of groundwater monitoring under the Radiation Protection Program included in FSAR Section 12AA.5.4.14 or describe why it is not necessary to update the FSAR with this information. The applicant stated that it added the information in FSAR Section 12AA.5.4.14 to FSAR Section 2.4.12.4 by reference (ML092150960). The staff reviewed the applicant's response and determined that the content of the referenced information is sufficient to address this information need.

2.4.12.4.8 Site Characteristics for Subsurface Hydrostatic Loading

Information Submitted by the Applicant

The applicant stated that the nominal plant grade elevation for the LNP site as 15.2 m (50 ft) NAVD88 and the nominal plant grade floor elevation for LNP 1 and LNP 2 as 15.5 m (51 ft) NAVD88. The AP1000 DCD indicates that the AP1000 is designed for a groundwater elevation up to 14.6 m (48 ft) NAVD88, which is 0.6 m (2 ft) below the nominal plant grade.

The applicant stated that twice daily groundwater elevation measurements recorded every 12 hours by pressure transducers in monitoring wells MW-13S and MW-15S, both completed in the surficial aquifer, resulted in maximum observed water levels during March 2007 and March 2008 that were more than 2.1 m (7 ft) below nominal plant grade elevation. This maximum observed water level corresponds to a water table elevation of 13.1 m (43 ft) NAVD88. The highest groundwater levels measured during quarterly monitoring events were 12.82 m (42.05 ft). These measurements were also at surficial aquifer wells MW-13S and MW-15S.

The applicant stated that "final grading of the LNP site will result in potential hydrologic alteration, including the permanent change in groundwater levels within the plant site from site grading and a series of stormwater drainage ditches…. Stormwater drainage ditches installed within the LNP site will have bottom elevations ranging from approximately 12.97 m (42.55 ft) NAVD88 or lower to approximately 14.57 m (47.80 ft) NAVD88." The applicant concluded that the LNP site meets the requirements for the AP1000 design and that "no dynamic water forces associated with normal groundwater levels will occur because of a higher finished plant grade."

NRC Staff's Technical Evaluation

The staff issued RAI 2.4.12-18 asking the applicant to provide an analysis and description of predicted post-construction groundwater conditions near the safety-related SSCs with respect to the DCD maximum allowable groundwater elevation. The applicant responded by reiterating the information in LNP FSAR Revision 2 concerning monitored water levels in comparison to the plant grade (ML092150960). The applicant referred to a calculation package concerning the effect of grouting on groundwater flow. The staff reviewed this calculation package and determined that it did not address the issue of expected groundwater level during plant operation. The applicant also referred to the response to RAI 2.4.12-02, which describes the results of a revision to the site groundwater model documented by the applicant (ML092240668). However, this model was revised by the applicant as documented by the applicant (ML093620211). The revised groundwater model shows that pumping of the water supply wells may create a drawdown of about 0.15 m (0.5 ft) at the LNP Unit 1 and Unit 2 plant locations.

As a follow-up to the applicant's response to RAI 2.4.12-18, the staff issued RAI 2.4.12-24 asking the applicant to analyze and describe the effects of alterations to the groundwater flow system, including the effects of stormwater runoff caused by the new structures and facilities and how this will affect groundwater levels near the safety-related SSCs with respect to the DCD maximum allowable groundwater elevation.

The applicant responded to RAI 2.4.12-24 by providing descriptions of alterations to the groundwater flow system and a discussion of the potential effects of each alteration on future groundwater elevations with respect to subsurface hydrostatic loading on LNP Unit 1 and LNP Unit 2 (ML101740492). The applicant will install a drainage system designed to remove runoff from up to a 50-year precipitation event. The applicant described that "the drainage system will capture and redirect rainfall and surface runoff away from safety-related SSCs to onsite ditches and retention ponds where the water will recharge, evaporate, or be pumped offsite if needed (via the cooling water tower basins)." The applicant stated that surficial aquifer groundwater elevations near safety-related SSCs would be reduced as a result of the drainage system. The applicant also stated that "if the onsite drainage system becomes blocked, the LNP site can be drained by overland flow directly to the Lower Withlacoochee River or the Gulf of Mexico." The applicant also described changes to the groundwater flow system resulting from the installation of impervious surfaces such as buildings and parking lots. The applicant stated that these impervious surfaces would result in less infiltration and reduce the potential for groundwater mounding around the safety-related SSCs during rainfall events. The applicant described planned grading of the site to drain surface flow away from the safety-related SSCs. The applicant described the planned dewatering system that will be used to lower groundwater levels around the nuclear islands during foundation emplacement and referred to a calculation package that was reviewed by the staff.

The staff issued RAI 2.4.12-25 asking the applicant to provide an estimate of the maximum post-construction groundwater level that is based on anticipated post-construction surface conditions, the anticipated properties of the fill material, the conceptual model of the subsurface, and expected maximum recharge rates. The applicant was also requested to provide proposed updates to the FSAR that would include the results of this analysis and supporting information used in the analysis.

The applicant responded by: (1) describing the planned installation of diaphragm walls at the excavation limits of the nuclear islands and grouting at the base of the excavations; (2) describing the surface grading and storm drainage system that is designed to direct stormwater and groundwater away from LNP Unit 1 and LNP Unit 2; and (3) providing the results of MODFLOW groundwater modeling performed to evaluate the maximum water table elevation (ML110800090). This modeling is distinct from the original and revised models used to investigate potential effects of groundwater usage, as described in Section 2.4.12.4.5 of this SER.

The staff reviewed the local groundwater model provided by the applicant and made independent model runs to confirm the applicant's conclusions and, in addition, to investigate the sensitivity of the model to certain parameters. Model input files were obtained from the applicant and the model parameters, boundary conditions, and results were verified. The groundwater model simulated the water table response under conditions of a 72-hr duration PMP design storm. The model divided the LNP site into specified areas of impervious surface material with no recharge of precipitation to the aquifer and areas of pervious materials that would experience a varying recharge rate calculated based on the hourly PMP precipitation rate. Three layers were implemented in the model. The top layer representing the surficial aquifer was assigned a uniform horizontal hydraulic conductivity of 2.8 m/d (9.2 ft/d) and a vertical hydraulic conductivity of 0.28 m/d (0.92 ft/d). Layers 2, 3, and 4 represented the Upper Floridan aquifer and were assigned a horizontal hydraulic conductivity of 4.2 m/d (13.9 ft/d) and vertical hydraulic conductivity of 0.4 m/d (1.39 ft/d). The horizontal hydraulic conductivity values applied to the Upper Floridan aquifer are significantly lower than the range of 36.6 to 39.6 m/d (120 to 130 ft/d) for the hydraulic conductivity determined from the MLU analyses of the applicant's pumping test. The value applied to the surficial aquifer is within the range of 0.27 to 8.72 m/d (0.9 to 28.6 ft/d) from the applicant's analysis of slug tests in the surficial aquifer. The staff determined that applying a relatively low hydraulic conductivity to the Upper Floridan aquifer model layer was conservative with regard to maximum water table elevation because a higher hydraulic conductivity would result in less mounding of the water table in response to infiltration of precipitation.

Recharge rates applied to the pervious areas of the model were calculated based on the average PMP precipitation rate during each model time step. The staff review of the model files showed that of a total of 90.7 cm (35.7 in.) of water recharged the upper layer of the model in pervious surface areas during the simulated PMP storm compared to a total PMP precipitation of 90.9 cm (35.8 in). This high rate of infiltration is a conservative factor in the analysis.

The applicant's model showed that during a PMP event, the water table elevations at the SSCs are predicted to be less than 13.7 m (45 ft) NAVD88, which is well below the 14.6 m (48 ft) NAVD88 limit defined by the DCD. The SSCs are surrounded by areas of impervious surface materials. Runoff will be routed to the stormwater drainage ditches that have bottom elevations from 13 to 14.6 m (42.5 ft to 47.8 ft) NAVD88. Based on the model results, the staff concludes that the maximum groundwater level will likely not exceed the DCD-specified maximum of

14.6 m (48 ft) NAVD88 at the safety-related structures. The water table was predicted by the model to reach the ground surface elevation of 15.2 m (50 ft) NAVD88 in some areas covered with pervious materials during a PMP design storm. However, the staff concludes that excess precipitation will runoff to the stormwater ditches and ponds and will not create a potential for groundwater levels exceeding the DCD limit.

Planned installation of diaphragm walls at the excavation limits of the nuclear islands and grouting at the base of the excavations will also reduce the potential for the water table to exceed the DCD design limit within the excavation areas. The staff determined that the planned diaphragm walls will not retain groundwater after plant construction in a way that would cause groundwater levels around the plant foundations to exceed the DCD design limit.

The applicant committed to revising the FSAR to include a description of the local-scale groundwater model and results related to estimating the expected maximum water table at safety-related structures. The staff is tracking this issue as **Confirmatory Item 2.4.12-1**.

Resolution of Confirmatory Item 2.4.12-1

Confirmatory Item 2.4.12-1 is an applicant commitment to update Section 2.4.12 of its FSAR. The staff verified that LNP COL FSAR Section 2.4.12 was appropriately updated. As a result, Confirmatory Item 2.4.12-1 is now closed.

2.4.12.5 *Post Combined License Activities*

There are no post COL activities related to this section.

2.4.12.6 *Conclusion*

The staff has reviewed the application and has confirmed that the applicant addressed the information relevant to groundwater, and that there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant presented and substantiated information to establish the site description. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.12 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses COL information item 2.4-4.

2.4.13 Accidental Release Of Radioactive Liquid Effluent In Ground And Surface Waters

2.4.13.1 *Introduction*

FSAR Section 2.4.13 provides a characterization of the attenuation, retardation, dilution, and concentrating properties governing transport processes in the surface water and groundwater environment at the site. This section's goal is not to assess the impacts of all possible specific release scenarios, but to provide a suitable conceptual model of the transport through the hydrological environment for possible later use in other assessments. Because it would be impractical to characterize all the physical and chemical properties (e.g., hydraulic conductivities, porosity, mineralogy) of a time-varying and heterogeneous environment, FSAR Section 2.4.13 characterizes the environment in terms of the projected transport of a postulated release of radioactive waste. The accidental release of radioactive liquid effluents in ground and surface waters is evaluated using information on existing uses of groundwater and surface water and their known and likely future uses as the basis for selecting a location to summarize the results of the transport calculation. The source term from a postulated accidental release is reviewed under NUREG-0800 (NRC 2007a) Section 11.2 following the guidance in Branch Technical Position (BTP) 11-6, "Postulated Radioactive Releases Due to Liquid-containing Tank Failures" (NRC 2007d). The source term is determined from a postulated release from a single tank outside of the containment. The tank having the greatest potential inventory of radioactive materials is assumed as the source of the release.

Section 2.4.13 of this SER presents an evaluation of the following specific areas: (1) alternative conceptual models of the hydrology at the site that reasonably bound hydrogeological conditions at the site inasmuch as these conditions affect the transport of radioactive liquid effluent in the groundwater and surface water environment; (2) a bounding set of plausible surface and subsurface pathways from potential points of an accidental release to determine the critical pathways that may result in the most severe impact on existing uses and known and likely future uses of groundwater and surface water resources in the vicinity of the site; (3) ability of the groundwater and surface water environments to delay, disperse, dilute, or concentrate accidentally released radioactive liquid effluents during transport; and (4) assessment of scenarios wherein an accidental release of radioactive effluents is combined with potential effects of seismic and non-seismic events (e.g., assessing effects of hydraulic structures located upstream and downstream of the plant in the event of structural or operational failures and the ensuing sudden changes in the regime of flow); and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.4.13.2 *Summary of Application*

This section of the COL FSAR addresses the accidental release of radioactive liquid effluents in groundwater and surface waters. The applicant addressed these issues as follows:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.4-5

This COL item is addressed by FSAR Section 2.4.13. In particular, this section addresses the following COL-specific information that is defined and discussed in Section 2.4.1.5 of Revision 19 of the AP1000 DCD.

Combined License applicants referencing the AP1000 certified design will address site-specific information on the ability of the ground and surface water to disperse, dilute, or concentrate accidental releases of liquid effluents. Effects of these releases on existing and known future use of surface water resources will also be addressed.

2.4.13.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the pathways of liquid effluents in groundwater and surface water, and the associated acceptance criteria, are described in Section 2.4.13 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements for liquid effluent pathways for groundwater and surface water are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 20, as it relates to effluent concentration limits.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Appropriate sections of the following documents are used for the related acceptance criteria:

- BTP 11-6 (NRC 2007d) provides guidance in assessing a potential release of radioactive liquids following the postulated failure of a tank and its components, located outside of containment, and impacts of the release of radioactive materials at the nearest potable water supply, located in an unrestricted area, for direct human consumption or indirectly through animals, crops, and food processing.

- Regulatory Guide 1.113, “Estimating Aquatic Dispersion of Effluents from Accidental and Routine Reactor Releases for the Purpose of Implementing Appendix I” (NRC 1977b)

2.4.13.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.13 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff’s review confirmed that the information in the application and incorporated by reference addresses the required information relating to accidental releases of radioactive liquid effluents in ground and surface waters. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

2.4.13.4.1 Radioactive Tank Rupture

Information Supplied by the Applicant

The applicant selected the accidental release to groundwater scenario based on information provided by the AP1000 reactor vendor. According to the applicant, the scenario is an instantaneous release from one of the two effluent holdup tanks located in the lowest level of the AP1000 auxiliary building. Each effluent holdup tank holds 105,992 L (28,000 gallons). The failed tank was assumed to have maximum radionuclide concentrations corresponding to101 percent of the reactor coolant source term. It was assumed that 80 percent of the tank's volume, or 84,793 L (22,400 gal) is released. The applicant provided the expected tank inventory in LNP FSAR Revision 2 Table 2.4.13-202. The applicant described the effluent holdup tanks as having the highest potential radionuclide concentration and the largest volume and, therefore, release from one of those tanks was considered a conservative selection for the purpose of calculating the potential for contamination of groundwater.

The applicant assumed that the effluent release occurs at the bottom floor of the auxiliary building and directly to the Floridan aquifer. No credit was taken for transit time through the walls of the auxiliary building, or through the surficial aquifer that overlies the Floridan aquifer. The bottom floor of the auxiliary building was described as 10.4 m (34 ft) below the design plant grade of 15.2 m (50 ft) elevation (NAVD88). The applicant considered a release directly to the Floridan aquifer to be conservative because the analysis does not take credit for transit time through the surficial aquifer and because the Floridan aquifer has higher seepage velocities than the surficial aquifer.

The applicant considered two transport cases. The first case was transport to a well completed in the Upper Floridan aquifer located on the LNP site boundary in the direction of groundwater flow at a distance of 2 km (1.2 mi). The second case considered groundwater transport to the Lower Withlacoochee River downgradient from LNP Units1 and 2 at a distance of approximately 6.9 km (4.3 mi).

The applicant determined the direction of groundwater flow to the southwest by examining observed groundwater head contour maps based on water levels measured in the onsite monitoring wells.

NRC Staff's Technical Evaluation

The staff reviewed the accidental release scenario and conceptual model. The tank rupture scenario was determined to be conservative because it assumes that 80 percent of the tank volume is instantaneously transmitted into the aquifer and this volume contains 101 percent of the coolant source term. The two transport cases are evaluated in the following section.

2.4.13.4.2 Groundwater Scenarios

Information Supplied by the Applicant

LNP FSAR Rev 2 stated that “The surficial aquifer is not a well-developed aquifer system near the LNP site and no users of surface water have been identified near the LNP site. … The Floridan aquifer is the principal source of potable water near the LNP site.” Therefore, the transport analysis was based on immediate release to the Floridan aquifer with no credit for transport time through the containment building or through the surficial aquifer.

The applicant calculated transport of radionuclides in groundwater using the analytical equation for three-dimensional, transient transport in a saturated porous medium with one-dimensional, steady advection in the x-direction, three-dimensional dispersion, linear equilibrium adsorption, and first-order decay. However, LNP FSAR Revision 2 states “The maximum concentration at a well in the Floridan aquifer is taken as the aquifer’s concentration at the distance downgradient from the point of release with vertical mixing assumed in the aquifer.” Therefore, the analysis assumes that the radionuclides are completely mixed over the assumed 76.2-m (250-ft) thickness of the aquifer.

The applicant identified key parameters used in radionuclide transport calculations. Seepage velocities used in the calculation were presented in Section 2.4.12 of LNP FSAR Rev 4. Distribution coefficients (K_d) for cesium and strontium were selected using EPA (1999) guidance for conservative selection of distribution coefficients. Other radionuclides were given K_d of zero, indicating no sorption. FSAR Rev. 4 references NUREG/CR-3332 (EPA 1983) to show that longitudinal dispersivity of αL = 10 to 15 m (32.8 to 49.2 ft) for limestone and carbonate aquifers are reasonable. However, the evaluation presented in FSAR Rev. 4 conservatively assumed longitudinal and transverse dispersivities of αL = 1 m and $\alpha L^*\alpha T$ = 1 m^2, respectively. Lower dispersivity values used in the analysis will result in higher concentrations of radionuclides at the receptor locations.

The LNP FSAR Revision 4 calculations of maximum activity concentrations in well water from a release to the Floridan aquifer resulted in an effective dose equivalent of less than 0.7 percent of the regulatory allowable activity. Tritium was found to be responsible for essentially the entire dose for water use derived from the well. The applicant also calculated radionuclide concentrations and resulting dose equivalents in the Lower Withlacoochee River. The calculated effective dose equivalent for the river water was negligible when compared to allowable limits.

NRC Staff’s Technical Evaluation

The staff issued RAI 2.4.13-02 asking the applicant to describe the process followed to ensure that the most conservative of plausible conceptual models were identified. The applicant responded with additional details concerning the identification of groundwater and surface water users, general site characteristics, and plausible surface and subsurface pathways (ML092080078). The most conservative conceptual models identified were (1) transport to a groundwater user located 2 km from the spill through the Upper Floridan aquifer with no credit for transport time through the containment building or through the surficial aquifer, and (2) contaminated groundwater entering the Withlacoochee River 7 km (4.3 mi) away from the spill also with no credit for transport time through the containment building or through the surficial aquifer.

The staff issued RAI 2.4.13-03 asking the applicant to clarify the total thickness of the Upper Floridan aquifer at the LNP site. The applicant responded by providing additional information about the thickness of the Upper Floridan aquifer above the MCU (ML092080078) and revised the FSAR discussion in Section 2.4.13.2. The applicant RAI response stated “Based on limited downhole geophysical testing and monitoring of drilling fluid losses at the LNP site, the most productive interval of the Upper Floridan aquifer appears to be at depths of approximately 30 to 60 m (100 to 300 ft) bgs.” However, 60 m would be equivalent to about 200 ft. The applicant used an aquifer thickness of 76.2 m (250 ft) in the assessment of an accidental release of radioactive effluents in groundwater. As a follow-up to the applicant’s response to RAI 2.4.13-03, the staff issued a new RAI 2.4.13-12 asking the applicant to clarify the apparent discrepancy regarding the depth of the most productive interval of the Upper Floridan aquifer. The applicant responded that the depth of 60 m is incorrect and the correct depth is 91 m, which corresponds to the 91.4-m (300-ft) value in FSAR Revision 2.

As a follow-up to RAI 2.4.13-02, the staff issued RAI 2.4.13-13 requesting that the applicant provide a discussion of the degree of conservatism in the transport analysis regarding (1) parameters used in seepage velocity calculations, (2) the assumption that the released contamination is evenly distributed over an aquifer thickness of 76.2 m (250 ft), and (3) the use of a groundwater head gradient in the transport analysis that is smaller than the gradient calculated from the potentiometric map for the Upper Floridan aquifer presented in the recalibrated version of the groundwater flow model (ML093620211), which is based on a more extensive well network. The applicant responded by describing a number of conservative assumptions in the analysis, including the receptor location on the site boundary and the direct release of effluent to the Upper Floridan aquifer (ML101830016). The applicant's response also discussed the hydraulic conductivity and effective porosity values, the aquifer thickness used in the analysis, and hydraulic gradients. Although the applicant defended the parameters and assumptions used in the FSAR analysis, the applicant also provided an “alternate evaluation” of groundwater transport through the Upper Floridan aquifer based on more conservative assumptions concerning aquifer hydraulic conductivity and effective porosity that reflect the potential for preferential flow paths within the fractured limestone aquifer. The parameters used in the alternate evaluation and the alternate transport analysis results, including the sum of fractions of the predicted concentration/Effluent Concentration Limits (ECL) reported in the RAI response, are listed below:

Alternate Analysis Parameters (different from original analysis):

- Hydraulic conductivity = 39.6 m/d (130 ft/d)
- Effective porosity = 0.05

Alternate Analysis Results:

- Linear velocity = 0.56 m/d (1.8 ft/d)
- Concentration/ECL – all nuclides = 54 percent (at offsite groundwater well)
- Peak time – tritium = 9.8 yr (at offsite groundwater well)
- Peak concentration – tritium = 5.2E-04 μCi/cm^3 (at offsite groundwater well)
- Concentration/ECL – tritium only = 52 percent (at offsite groundwater well)

The alternate transport analysis used the same aquifer thickness (76.2 m [250 ft]) and gradient as were used in the FSAR Revision 2 analysis.

The applicant also provided an analysis of vertical dispersion for comparison with the assumption of complete vertical mixing over the assumed 76.2 m (250 ft) aquifer thickness to address the staff concern. The analysis showed that for a contaminant not affected by decay or retardation, the vertical distribution of contaminant concentrations at the top and bottom of the 76.2-m (250-ft) aquifer are within 7 percent of "fully mixed" when the center of the plume has moved 2 km (1.24 mi) from the release point. The analysis was based on the parameters applied in the LNP FSAR Revision 2 transport calculations.

In the response to RAI 2.4.13-13 (ML101830016), the applicant compared groundwater gradients from onsite measurements to the potentiometric map for the Upper Floridan aquifer presented in the recalibrated version of the groundwater flow model (ML093620211). The potentiometric map was based on some wells located in an area of higher groundwater levels more than 4 mi northeast of the LNP site and on synthetic wells based on modeled USGS water level contours. The applicant presented the data to show that the gradient of 0.0007 used in transport modeling is at the upper range calculated from onsite well measurements for the direction of groundwater flow from the reactor locations toward the receptor well.

The staff reviewed the applicant's responses to RAI 2.4.13-02 (ML092080078) and RAI 2.4.13-13 (ML101830016) and determined that the release to groundwater scenarios for contaminant transport presented in the FSAR are conservative except with regard to values of saturated hydraulic conductivity (16.6 m/d [54.4 ft/d]) and effective porosity (0.15) used in the seepage velocity calculations. The staff determined that the applicant's "alternate evaluation" of groundwater transport through the Upper Floridan aquifer provides a conservative analysis of the pathway associated with an accidental spill to groundwater. The alternate analysis was based on a higher (more conservative) saturated hydraulic conductivity (39.6 m/d [130 ft/d]) from MLU analysis of the aquifer pumping test and a lower (more conservative) effective porosity (0.05) that reflects the possibility of preferential flow paths within the fractured limestone aquifer. Other parameters used in the alternate evaluation matched those used in the FSAR analysis.

The staff also reviewed the discussion and analysis of vertical dispersion provided in response to RAI 2.4.13-13 (ML101830016). The analysis showed that for a contaminant not affected by decay or retardation, the vertical distribution of contaminant concentrations at the top and bottom of the assumed 250-ft aquifer are within 7 percent of “fully mixed” when the center of the plume has moved 2 km (1.24 mi.) from the release point. The analysis was based on the parameters applied in the LNP FSAR Revision 2 transport calculations. The staff considers the analysis based on a contaminant not affected by decay or retardation to be appropriate because tritium is the primary dose contributor.

The staff issued RAI 2.4.13-04 asking the applicant to “discuss LNP groundwater usage from the Upper Floridan aquifer in relation to the projected impacts of pumping on subsurface radionuclide transport pathways at the LNP site.” Related RAIs, 2.4.12-02 and 2.4.12-24, asked the applicant to discuss the effects of alterations to the groundwater flow system, including details of plant water supply wells and the projected impacts of pumping on transport pathways, surrounding surface waters, and adjacent offsite groundwater users. The applicant responded (ML092080078) with additional information about the planned water supply wells and discussed the results of a site groundwater model (ML092240668). However, this model was subsequently revised by the applicant based on an RAI related to the LNP EIS. The new revision of the groundwater model was documented by the applicant (ML093620211). The applicant's revised groundwater flow model (ML093620211) predicts drawdown of 0.46 to 0.61 m (1.5 to 2 ft) in the southern portion of the LNP site after 1 year caused by operation of the water supply wells. This would result in a larger gradient to the south. A 0.6-m (2-ft) decrease in head near the water supply wells, about 2.4 km (1.5 mi) from the release point, would result in a gradient increase from 0.0007 to 0.00095 based on the revised model results. However, pumping at the supply wells would also result in a longer south-southwest flow path to the site boundary of about 3.2 km (2 mi), which would result in a slightly longer travel time than that calculated based on the gradient and flow path used in the LNP FSAR Revision 2 analysis.

The staff reviewed the applicant response to RAI 2.4.13-04 regarding the impact of groundwater usage from the Upper Floridan aquifer, including pumping of the proposed plant water supply wells on subsurface radionuclide transport pathways. The staff concurs that the water table may experience drawdown of 0.5 to 0.6 m (1.5 to 2 ft) in the southern portion of the LNP site after 1 year because of the water supply wells and this would result in a larger gradient to the south. However, the change in water table configuration would result in a longer south-southwest flow path to the site boundary of about 3.2 km (2 mi), which would result in a slightly longer travel time than that calculated based on the gradient and flow path used in the LNP FSAR Revision 2 analysis. The staff also agrees that the onsite measurements used by the applicant in gradient calculations are more representative of groundwater flow conditions along the hypothetical transport path than the potentiometric map for the Upper Floridan aquifer presented in the recalibrated version of the groundwater flow model (ML093620211), because the potentiometric map was based on some wells located in an area of higher groundwater levels more than 6.4 km (4 mi) northeast of the LNP site and on synthetic wells based on modeled USGS water level contours.

RAI 2.4.13-05 asked the applicant to discuss why assuming a release at the top of the Floridan aquifer is conservative and whether a release to the surficial aquifer could result in a pathway to surface water, such as the Withlacoochee River, and including marshes or ditches at the LNP site that are closer than the nearest offsite well. The applicant responded (ML092080078) by explaining that the release would occur about 7.6 m (25 ft) below the top of the surficial aquifer, and about 7.6 m (25 ft) above the top of the Floridan aquifer. Downward head gradients within the surficial aquifer would make radionuclides migrate downward to the Floridan aquifer. The applicant also provided additional information about the site topography and surface features and the planned surface water drainage system.

The staff concurs with the applicant's response to RAI 2.4.13-05 that a release to surface water is not likely because of the location of the release 10.4 m (34 ft) below the nominal plant grade elevation. The measured downward vertical hydraulic gradient would also make it unlikely that contaminants would migrate upward through the surficial aquifer. It is unlikely that contaminants would migrate from this depth to marshes or ditches at the LNP site that are closer than the nearest offsite well. RAI 2.4.13-06 stated that “PEF needs to clarify why use of the one-dimensional advection-dispersion equation for solute transport in porous media is appropriate at the LNP site.” The applicant responded (ML092080078) with additional information and references describing groundwater flow and transport characteristics expected for the Upper Floridan aquifer. The applicant presented evidence that groundwater flow between the LNP plant locations and an offsite receptor well is expected to be laminar and dispersive and follow Darcy's law. The applicant response also provided sensitivity calculations showing the effects of higher pore velocities (compared with those in Section 2.4.12 of FSAR Revision 1) on the total dose calculated at the hypothetical downgradient well.

The staff reviewed the applicant's response to RAI 2.4.13-06 regarding use of the one-dimensional advection-dispersion equation for solute transport in porous media. The staff agrees that groundwater flow between the LNP plant locations and an offsite receptor well is expected to be laminar and follow Darcy's law.

The staff issued RAI 2.4.13-07 asking the applicant to describe the computer software used to implement the mathematical model described in FSAR Section 2.4.13.2.1. Verification and validation procedures used to verify the accuracy of the model, as implemented in the software, were also requested. The applicant responded (ML092080078) by providing additional information about the calculation method, the Project Quality Plan and verification review procedures.

RAI 2.4.13-08 asked the applicant to list the sources of the model parameters listed in FSAR Table 2.4.13-203. The applicant response (ML092080078) provided a table listing the requested model parameters and notes with information about the sources. The applicant revised the FSAR by substituting the new Table 2.4.13-203.

The staff issued RAI 2.4.13-09 asking the applicant to provide the tritium concentration as a function of time in the FSAR, or justify why this information is not necessary. The applicant responded (ML092080078) by stating that “Because the evaluation for meeting 10 CFR 20 criteria is made using the maximum nuclide concentrations, the criteria is satisfied for all other times.” These maximum calculated nuclide concentrations are shown in the FSAR. The applicant's response also included plots of tritium concentration over time from the transport calculations and noted that almost the entire dose at the receptor locations is caused by tritium. The applicant also noted that the sum of all of the ratios of radionuclide concentrations to concentration limits are also provided in the FSAR to demonstrate that the criteria for mixtures are satisfied. The applicant made minor wording changes to the FSAR discussion in Section 2.4.13.2. The staff agrees that the radionuclide concentrations over time do not need to be shown in the FSAR as long as the maximum concentration over time is stated and is used in the evaluation for meeting the 10 CFR 20 criteria.

In RAI 2.4.13-10, the staff requested that the applicant provide site-specific measurements of K_d as required by 10 CFR 100.20(c)(3). The applicant had used literature-based values of K_d for the transport analysis described in FSAR Revision 2. In a letter dated July 22, 2009, the applicant provided laboratory measurements of K_d values on 16 soil and rock samples from the site. The applicant showed that using the site-specific K_d values in the transport analysis did not significantly change the results of the transport calculations. The applicant revised the FSAR by adding information about the site-specific K_d measurements.

The staff issued RAI 2.4.13-11 asking the applicant to discuss the potential impacts of chelating agents on K_d values and on radionuclide transport in the FSAR. In response to RAI 2.4.13-11, the applicant stated that only cesium and strontium were given non-zero K_d in the transport calculation. The applicant provided evidence from the literature that the transport behavior of cesium is not likely to be strongly influenced by chelating agents. The applicant also stated that cesium and strontium are unlikely to form complexes with chelating agents in groundwater because of the abundance of competing calcium and magnesium ions (ML092080078). The staff reviewed this information and determined that, based on the evidence for minor influence of chelating agents on cesium and strontium behavior in the groundwater and minor impact on the calculated sum of radionuclides at the receptor locations, the applicant's response meets this information need.

The staff evaluation confirmed that assuming immediate release to the Upper Floridan aquifer with no credit for transport time through the containment building or through the surficial aquifer was a conservative assumption. This pathway is the most conservative of the plausible pathways discussed in Section 2.4.12. The hypothetical release occurs about 7.6 m (25 ft) below the top of the surficial aquifer and 7.6 m (25 ft) above the top of the Upper Floridan aquifer. The measured downward vertical flow gradient makes it unlikely that contaminants will migrate upward to wetlands or other receptors at the ground surface. The applicant did not take credit for time required for released contaminants to migrate from inside the auxiliary building through the surficial aquifer sediments or through the diaphragm wall that will extend about 30 ft into the pressure grouted limestone at the top of the Upper Floridan aquifer (LNP FSAR Revision 4 Section 2.5.4.6. The diaphragm walls are specified to be a minimum of 1.1 m (3.5 ft) thick. The staff checked site borehole logs to verify that there is approximately 7.6 m (25 ft) of surficial aquifer sediment below the release elevation and above the top of the Upper Floridan aquifer.

To summarize, the staff reviewed the transport calculation equations provided in LNP FSAR Rev 2 and determined that they are consistent with the solutions given in NUREG/CR-3332 Section 4.5.3 (EPA 1983). The values used by the applicant for K_d and dispersivity parameters were found to be conservative estimates for the Upper Floridan aquifer. However, the seepage velocity values used in the transport calculations were found to not be conservative in the analysis presented in LNP FSAR Revision 2. These issues were addressed in RAIs issued to the applicant and ultimately resulted in the applicant providing an "alternate analysis" of groundwater transport through the Upper Floridan aquifer based on more conservative assumptions concerning aquifer hydraulic properties.

The staff determined that the applicant's "alternate analysis" of groundwater transport provided in response to RAI 2.4.13-13 (ML101830016) presents a conservative calculation of the potential dose impacts from a release of radioactive liquid effluent to groundwater. The hydraulic conductivity and effective porosity values used in the alternative analysis are conservative yet conceivable estimates of the conditions found in this portion of the Upper Floridan aquifer. The selected pathway through the Upper Floridan aquifer to a groundwater user is the most conservative of the reasonably foreseeable pathways based on the available site data. Although there is uncertainty in some of the parameters used in the analysis and more conservative parameter values are possible, the very conservative assumption of not accounting for migration time through the containment building, the diaphragm walls and grouted limestone, or the 7.6-m (25-ft) thickness of surficial aquifer, through which radionuclides would migrate downward, results in calculated travel times that are bounding. Including transport through the dewatering structure would result in travel times more than double those calculated in the alternative analysis. The assumption of complete mixing of contaminants over the aquifer thickness is not conservative, but the applicant has demonstrated that the predicted radionuclide concentrations at the offsite receptor location will be less than 10 percent lower than the values calculated using a vertical dispersion model. This is compensated by use of a 76.2-m (250-ft) rather than a 91.4-m (300-ft) aquifer thickness.

2.4.13.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.13.6 *Conclusion*

The staff has reviewed the application and has confirmed that the applicant addressed the relevant information and there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant presented and substantiated information to establish the potential effects of accidental releases from the liquid waste management system. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description, and about the design of the liquid waste management system, for the staff to determine, as documented in Section 2.4.13 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site, and with respect to 10 CFR 20 as it relates to effluent concentration limits. This addresses COL information item 2.4-5.

2.4.14 Technical Specifications and Emergency Operation Requirements

2.4.14.1 *Introduction*

FSAR Section 2.4.14 of the LNP COL application describes the technical specifications and emergency operation requirements as necessary. The requirements described implement protection against floods for safety-related facilities to ensure that an adequate supply of water for shutdown and cool-down purposes is available.

Section 2.4.14 of this SER presents an evaluation of the following specific areas: (1) control of hydrological events, as determined in previous hydrology sections of the FSAR, to identify the bases for emergency actions required during these events; (2) the amount of time available to initiate and complete emergency procedures before the onset of conditions while controlling hydrological events that may prevent such action; (3) review of technical specifications related to all emergency procedures required to ensure adequate plant safety from controlling hydrological events by the organization responsible for the review of issues related to technical specifications; (4) potential effects of seismic and non-seismic information about the postulated technical specifications and emergency operations for the proposed plant site; and (5) any additional information requirements prescribed in the “Contents of Application” sections of the applicable subparts to 10 CFR Part 52.

2.4.14.2 *Summary of Application*

This subsection of the COL FSAR addresses technical specifications and emergency operation requirements. The applicant addressed the information as follows:

AP1000 COL Information Item

- LNP COL 2.4-6

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.6 of Revision 19 of the AP1000 DCD.

> Combined License applicants referencing the AP1000 certified design will address any flood protection emergency procedures required to meet the site parameter for flood level.

2.4.14.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for consideration of emergency protective measures, and the associated acceptance criteria, are described in Section 2.4.14 of NUREG-0800 (NRC 2007a).

The applicable regulatory requirements are as follows:

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 50.36, as it relates to identifying technical specifications related to all emergency procedures required to ensure adequate plant safety from controlling hydrological events by the organization responsible for the review of issues related to technical specifications.

2.4.14.4 *Technical Evaluation*

The NRC staff reviewed Section 2.4.14 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of the DCD and the COL application represents the complete scope of information relating to this review topic.[1] The NRC staff's review confirmed that the information in the application and incorporated by reference addresses the required information relating to technical specifications and emergency operation requirements. The results of the NRC staff's evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

Information Submitted by the Applicant

The applicant stated that the AP1000 design does not have a safety-related cooling-water system. The applicant also stated that flooding of the safety-related facilities is not a concern at the LNP site. The applicant concluded that no emergency protective measures are needed at the LNP site for hydrology-related adverse events.

NRC Staff's Technical Evaluation

The NRC staff has concluded in previous sections of this SER that floods caused by natural phenomena at and near the LNP site would not result in inundation of the plant grade. The AP1000 design does not use a safety-related cooling-water system. Therefore, the staff concluded that no technical specification or emergency procedures related to hydrologic events are required at the LNP site.

2.4.14.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.4.14.6 *Conclusion*

The staff reviewed the application and confirmed that the applicant has addressed the information relevant to technical specification and emergency operations requirements, and there is no outstanding information required to be addressed in the COL FSAR related to this section.

As set forth above, the applicant has presented and substantiated site-specific information related to technical specifications and emergency operations. The staff has reviewed the information provided and, for the reasons given above, concludes that the applicant has provided sufficient details about the site description for the staff to determine, as documented in Section 2.4.14 of this SER, that the applicant has met the relevant requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR Part 100 with respect to determining the acceptability of the site. This addresses COL Information Item 2.4-6.

2.5 Geology, Seismology, and Geotechnical Engineering

In Section 2.5, "Geology, Seismology, and Geotechnical Engineering," of the Levy Nuclear Plant (LNP) Units 1 and 2 Final Safety Analysis Report (FSAR), the applicant described geologic, seismic, and geotechnical engineering characteristics of the proposed combined license (COL) site. Following the U.S. Nuclear Regulatory Commission (NRC) guidance in Regulatory Guide (RG) 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)," and RG 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion," the applicant defined the following four zones around the LNP COL site and conducted technical investigations in these zones:

Site region – Area within a 320-kilometer (km) (200-mile (mi)) radius of the site location.
Site vicinity – Area within a 40-km (25-mi) radius of the site location.
Site area – Area within an 8-km (5-mi) radius of the site location.
Site location – Area within a 1-km (0.6-mi) radius of proposed LNP Units 1 and 2.

The applicant referred to the FSAR prepared by Florida Power Corporation (Florida Power Corporation, 1976) for the Crystal River Unit 3 Nuclear Generating Plant (CR3), located about 18 km (11 mi) southwest of the LNP COL site, to provide limited information deemed pertinent for understanding the geologic setting of the LNP site, particularly in regard to karst development. However, most material in Section 2.5 of the LNP COL FSAR draws on information developed from sources published since the CR3 site's FSAR, as well as data derived from geologic, seismic, and geotechnical engineering investigations performed specifically for characterization of the LNP site.

The applicant used seismic source models previously published by the Electric Power Research Institute (EPRI, 1986 and 1989) as the starting point for characterizing potential regional seismic sources and vibratory ground motion resulting from those sources. The applicant then updated these EPRI seismic source models in light of more recent data and evolving knowledge. The applicant also replaced the original EPRI ground motion models (EPRI, 1989) with more recent EPRI models (EPRI, 2004), and then applied the performance-based approach described in RG 1.208 to develop the ground motion response spectra (GMRS) for the LNP site. The applicant revised its original GMRS calculations presented in LNP COL Revisions 1 through 4 by scaling up the original GMRS by a factor of 1.212. This scaling factor is the same factor applied to the foundation input response spectra (FIRS) in compliance with the requirement in 10 CFR Part 50, Appendix S, that the horizontal component of the FIRS in the free-field at the foundation level of the structure be a response spectrum with a minimum PGA of 0.1g.

In addition, to address recommendations of the Fukushima Near-Term Task Force described in SECY-12-0025 and evaluate potential seismic hazards at the LNP site in light of these recommendations, the applicant performed sensitivity studies using the central and eastern United States seismic source characterization (CEUS SSC) model presented in NUREG-2115.

The GMRS calculated using the CEUS SSC model combined with the updated cumulative absolute velocity (CAV) filter methodology, as described in SECY-12-0025, is enveloped by the scaled GMRS based on the updated EPRI-SOG model with full CAV, except the maximum exceedance of 4 percent near 1 Hz.

As discussed further in SER Section 20.1, based on its review of the applicant's two seismic hazard evaluations using the EPRI-SOG model and CEUS SSC model using the updated CAV filter, the staff concludes that the LNP GMRS, FIRS, and performance based soil response spectra (PBSRS) calculated by the applicant using the CEUS SSC model are either bounded by the respective spectra calculated by the applicant using the updated EPRI-SOG model, or are within a range of percentage error expected for those calculations. Therefore, it is not necessary for the applicant to update the UHRS, GMRS, FIRS, and PBSRS calculated using the updated EPRI-SOG model.

This safety evaluation report (SER) for Section 2.5 is divided into five main parts, SER Sections 2.5.1 through 2.5.5, which parallel the five FSAR sections prepared by the applicant for the LNP COL application. The five SER sections are Section 2.5.1, "Basic Geologic and Seismic Information"; Section 2.5.2, "Vibratory Ground Motion"; Section 2.5.3, "Surface Faulting"; Section 2.5.4, "Stability of Subsurface Materials and Foundations"; and Section 2.5.5, "Stability of Slopes" (including information regarding embankments and dams). These SER sections present the staff's evaluations and conclusions in regard to the geologic, seismic, and geotechnical engineering characteristics for proposed LNP Units 1 and 2.

2.5.1 Basic Geologic and Seismic Information

2.5.1.1 *Introduction*

LNP COL FSAR Section 2.5.1 describes the basic geologic and seismic information collected by the applicant during site characterization investigations. This information addresses both regional and site-specific geologic and seismic characteristics. The investigations included surface and subsurface field studies, performed by the applicant at progressively greater levels of detail closer to the site within each of four circumscribed areas, which correspond to site region, site vicinity, site area, and site location, as previously defined. The applicant conducted these investigations to assess geologic and seismic suitability of the site; determine whether new geologic or seismic data exist that could significantly impact seismic design based on the results of probabilistic seismic hazard analysis (PSHA); and to provide the geologic and seismic data appropriate for plant design.

2.5.1.2 *Summary of Application*

Section 2.5 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.5.1 of the AP1000 Design Control Document (DCD), Revision 19.

In addition, in LNP COL FSAR Section 2.5.1, the applicant provided site-specific supplemental information to address the following:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.5-1

The applicant provided additional information in LNP COL 2.5-1 to address COL Information Item 2.5-1 (COL Action Item 2.5.1-1). LNP COL 2.5-1 addresses the provision of regional and site-specific geologic, seismic, and geophysical information, as well as conditions caused by human activity. This information specifically includes the following topics: structural geology; seismicity; geologic history; evidence of paleoseismicity; site stratigraphy and lithology; engineering significance of geologic features; site groundwater conditions; dynamic behavior during prior earthquakes; zones of alteration, irregular weathering, or structural weakness; unrelieved residual stresses in bedrock; materials that could be unstable because of mineralogy or physical properties; and the effects of human activities in the area.

LNP COL FSAR Section 2.5.1 is divided into two main sections. FSAR Section 2.5.1.1, “Regional Geology,” describes physiography and topography; geologic history; stratigraphy, including general characteristics of carbonate terrain; and tectonic setting, including seismicity, within the LNP site region. FSAR Section 2.5.1.1 also discusses significant seismic sources outside the site region. FSAR Section 2.5.1.2, “Site Geology,” addresses physiography and topography, including characteristics of marine terraces and karst terrain; geologic history; stratigraphy, including carbonate units and karst phenomena; and structural geology within the LNP site vicinity and site area. FSAR Section 2.5.1.2 also discusses geomorphology and stratigraphy, including karst development, at the site location, and evaluates geologic hazard and engineering geology of the site area and site location, respectively.

The applicant developed LNP COL FSAR Section 2.5.1 based on information derived from maps and reports published by state and federal agencies and research workers; remote sensing imagery and aerial photographs; digital elevation models (DEMs); oil and gas exploration programs; communications with researchers familiar with previous investigations in the site region, site vicinity, and site area; and geologic and geotechnical field studies performed specifically for characterization of the LNP site location, site area, and site vicinity. The applicant also provided limited information deemed pertinent for understanding the geologic setting of the LNP site, particularly in regard to karst development, as derived from the CR3 FSAR (Florida Power Corporation, 1976).

Based on the geologic and seismic investigations performed for LNP Units 1 and 2, the applicant concluded in FSAR Section 2.5.1 that no geologic or seismic conditions exist at the site, which would negatively impact the construction or operation of safety-related structures. The applicant further concluded that possible non-tectonic surface deformation related to dissolution of carbonate and resultant collapse or subsidence is the only potential geologic hazard in the site area, and that this hazard will be mitigated either during construction or by appropriate design. A summary of the basic geologic and seismic information the applicant provided in LNP COL FSAR Section 2.5.1 is presented below.

2.5.1.2.1 Regional Geology

FSAR Section 2.5.1.1 discusses the physiography and topography, geologic history, stratigraphy, and tectonic setting of the LNP site region, defined as that area which lies within a 320-km (200-mi) radius of the site. In the discussion of regional tectonic setting, the applicant also addressed regional seismicity and significant seismic sources at a distance greater than 320 km (200 mi) from the site. The following sections summarize the information the applicant provided in FSAR Section 2.5.1.1.

2.5.1.2.1.1 Regional Physiography and Topography

FSAR Section 2.5.1.1.1 describes physiography and topography of the Coastal Plain physiographic province in the site region, including the Sea Island, East Gulf, and Floridian sections of that physiographic province. SER Figure 2.5.1-1 (reproduced from FSAR Figure 2.5.1-201) shows the location of the LNP site in relation to these three sections of the Coastal Plain physiographic province, the Florida peninsula, and the Floridian plateau. The region containing the Floridian plateau and the Florida peninsula separates the Gulf of Mexico from the Atlantic Ocean and makes up the Florida platform. The LNP site lies on the Gulf side of the Florida peninsula, atop the Florida platform, in the Floridian section of the Coastal Plain physiographic province.

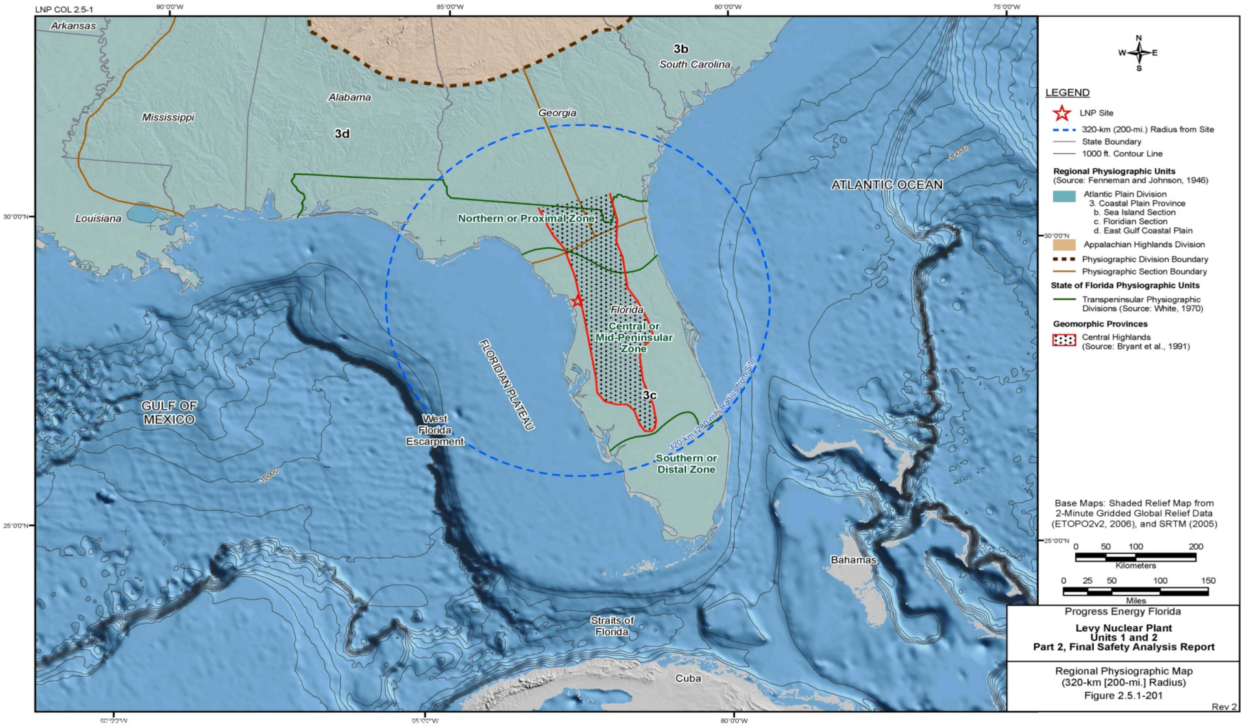


Figure 2.5.1-1. Regional Physiographic Map Showing Location of the LNP Site
(FSAR Figure 2.5.1-201)

In FSAR Section 2.5.1.1.1.1.1, the applicant stated that the Sea Island section of the Coastal Plain province (3b in SER Figure 2.5.1-1) is a youthful to mature terraced surface with a slightly submerged margin. In FSAR Section 2.5.1.1.1.1.2, the applicant described the East Gulf section of the Coastal Plain province (3d in SER Figure 2.5.1-1) as a youthful to maturely dissected region, consisting of alternating asymmetric ridges and lowlands with terraces along its outer margin.

In FSAR Section 2.5.1.1.1.1.3, the applicant noted that the Floridian section of the Coastal Plain physiographic province in which the LNP site is located encompasses the entire Florida peninsula (3c in SER Figure 2.5.1-1). The applicant reported that the Floridian section is a recent emergent platform characterized by widespread carbonate rocks with associated karst features. The Floridian section contains the Florida Keys along the southern tip of the Florida peninsula. Three physiographic zones comprise the Florida peninsula, namely the northern (proximal), central (midpeninsular), and southern (distal) zones. The LNP site lies in the midpeninsular zone as shown in SER Figure 2.5.1-1. Discontinuous subparallel ridges, oriented parallel to the length of the Florida peninsula and rising to about 61 meters (m) (200 feet (ft)) above mean sea level (amsl) and separated by broad valleys that may contain shallow lakes, comprise the midpeninsular zone.

2.5.1.2.1.2 Regional Geologic History

FSAR Section 2.5.1.1.2 discusses Late Proterozoic (> 542 million years in age, or Ma), Paleozoic (542 to 251 Ma), Mesozoic (251 to 65.5 Ma), and Cenozoic (65.5 Ma to present) geologic history of the LNP site region.

Late Proterozoic, Paleozoic, and Mesozoic Geologic History

The applicant summarized Late Proterozoic and Paleozoic geologic and tectonic history of the broad region containing the LNP site in FSAR Section 2.5.1.1.2.1. The applicant indicated that breakup of a supercontinental land mass by extensional rifting occurred around Late Proterozoic-Cambrian time (> 488 Ma), and that stratigraphic evidence shows several later compressional events, which culminated in formation of the Appalachian orogen at the end of the Paleozoic (251 Ma).

Regarding Mesozoic geologic and tectonic history, the applicant stated in FSAR Section 2.5.1.1.2.2 that rifting initiated during Triassic and Jurassic time (251 to 145.5 Ma) created the present-day Atlantic Ocean, and that the Gulf of Mexico formed completely by the end of the Jurassic (145.5 Ma). The applicant indicated that, since the end of extensive Triassic and Jurassic rifting, the entire Florida platform has been tectonically quiet based on the occurrence of undisturbed Upper Cretaceous (145.5 to 65.5 Ma) and Tertiary (65.5 to 2.6 Ma) strata on the platform.

Cenozoic Geologic History

In FSAR Section 2.5.1.1.2.3, the applicant stated that, during the first 35 million years of Cenozoic (65.5 Ma to present) time, sea levels were high and carbonate sedimentation dominated deposition on the Florida platform. The applicant noted that encroachment of clastic sediments onto the platform occurred slowly, with these sediments dominating deposition patterns on the platform during late Miocene to Pliocene (11.6 to 5.3 Ma) time. The applicant indicated that periodic regressions of the sea during the Miocene (23 to 5.3 Ma), Pliocene (5.3 to 2.6 Ma), and Quaternary (2.6 Ma to present) exposed vast areas of the carbonate platform, allowing karst features to develop. The applicant also stated that high sea-level stands occurred during the Pleistocene (2.6 Ma to 10,000 years) in southern Florida, and that no evidence exists in the Florida Keys to suggest any significant subsidence, uplift, or tectonic deformation of late Quaternary deposits.

2.5.1.2.1.3 Regional Stratigraphy

FSAR Section 2.5.1.1.3 describes stratigraphic relationships for pre-Cretaceous (> 145.5 Ma), Cretaceous (145.5 to 65.5 Ma), and post-Cretaceous (< 65.5 Ma) rock units, which occur in the LNP site region. The applicant stated that the low relief of the Florida peninsula reflects the nearly horizontal attitude of the predominately Cretaceous and Cenozoic (65.5 Ma to present) carbonate section, which underlies the peninsula and overlies pre-Cretaceous basement rocks of variable age and composition.

2.5.1.2.1.3.1 Pre-Cretaceous Stratigraphy

In FSAR Section 2.5.1.1.3.1, the applicant described the basement rocks which pre-date and underlie the Cretaceous (145.5 to 65.5 Ma) stratigraphic section at depth in the site region. These rocks are primarily Jurassic (201.6 to 145.5 Ma) igneous and volcaniclastic rocks in south Florida; Paleozoic (542 to 251 Ma) igneous and metamorphic rocks in central Florida; relatively undeformed Paleozoic sedimentary rocks in northern Florida; and faulted Paleozoic sedimentary units, which are covered by Triassic (251 to 201.6 Ma) sedimentary rocks, in the Florida panhandle.

2.5.1.2.1.3.2 Cretaceous and Post-Cretaceous Stratigraphy

In FSAR Section 2.5.1.1.3.2, the applicant indicated that Cretaceous (145.5 to 65.5 Ma) and post-Cretaceous (i.e., Cenozoic, 65.5 Ma to present) sedimentary strata of the Coastal Plain unconformably (i.e., representing a gap in the geologic record rather than continuous deposition) overlie pre-Cretaceous (> 145.5 Ma) basement rocks in Florida and adjacent areas of Alabama and Georgia. These strata, deposited in a relatively stable tectonic environment, consist of nearly flat-lying marine units approximately 7 km (4 mi) thick that terminate at the escarpments bounding the Florida platform. This stratigraphic section generally exhibits a west-to-east and north-to-south gradation from clastic to carbonate units.

The applicant reported a striking lithologic contrast between strata of peninsular Florida, which are primarily carbonates, and the predominantly clastic rocks of the Florida panhandle. The middle Eocene (48.6 to 40.4 Ma) Avon Park Formation, the oldest exposed rock unit in Florida, is a carbonate sequence that underlies all of peninsular Florida and forms the foundation unit for the LNP site. The formation exhibits pervasive dolomitization of some stratigraphic horizons (i.e., pure limestone of the Avon Park, made up of calcium carbonate, has been altered to dolomite, calcium magnesium carbonate, by magnesium-bearing waters), and it contains interbedded evaporite deposits (i.e., sedimentary rock units composed mainly of minerals produced from saline solutions as a result of extensive evaporation of the solvent fluid) in its lower part.

2.5.1.2.1.4 Regional Tectonic Setting

FSAR Section 2.5.1.1.4 discusses tectonic setting of the site region. The applicant addressed contemporary tectonic stress; structural setting and geophysical framework as defined by gravity and magnetic data; regional tectonic structures; significant seismic sources at a distance greater than 320 km (200 mi) from the LNP site; and regional seismicity. The applicant specifically assessed major Paleozoic, Mesozoic, and Cenozoic tectonic structures and concluded that none of these regional features are capable tectonic structures.

2.5.1.2.1.4.1 Contemporary Tectonic Stress

In FSAR Section 2.5.1.1.4.1, based on Zoback and Zoback (1989), the applicant indicated that a relatively uniform east-northeast compressive stress field extends regionally from the midcontinent eastward toward the Atlantic continental margin, and that no available data support a distinct Atlantic Coastal Plain stress province. The applicant cited Zoback and Zoback (1980) to suggest that southward-oriented extension along the northern Gulf of Mexico region reflects crustal loading and deformation within the Mississippi River delta complex, rather than effects of the regional east-northeast compressive stress field. The applicant cited Crone and others (1997) to classify the site region as a stable continental region (SCR), and characterized the region as exhibiting low earthquake activity and low stress based on Johnston and others (1994).

2.5.1.2.1.4.2 Regional Structural Setting and Geophysical Framework

In FSAR Section 2.5.1.1.4.2, the applicant stated that continental crust modified by Middle Jurassic (176 to 161 Ma) or later extensional rifting underlies the LNP site at depth. The site lies on the Florida platform near the northeastern margin of the Gulf Coast basin, and the applicant noted that this basin contains sedimentary strata up to 15 km (9 mi) thick, which overlie basement and range in age from Late Triassic (235 to 201.6 Ma) to Holocene (10,000 years to present). Based on Smith and Lord (1997), the applicant indicated that these strata contribute little to regional gravity and magnetic anomalies. The applicant attributed the marked contrast in gravity and magnetic anomalies between southern and northern Florida to a major change in composition of crustal basement from oceanic crust beneath southern Florida to continental crust beneath northern Florida. The applicant commented that this disparity in gravity and magnetic anomalies between northern and southern Florida has been postulated as evidence for a regional basement fault beneath peninsular Florida, which developed during Jurassic (201.6 to 145.5 Ma) time. The applicant noted that Smith and Lord (1997) referred to this basement feature as the Jay fault, or Florida lineament, and interpreted it to represent the northwestern extension of the Bahamas fracture zone across southern Florida.

2.5.1.2.1.4.3 Regional Tectonic Structures

In FSAR Section 2.5.1.1.4.3, the applicant discussed regional tectonic structures within a 320-km (200-mi) radius of the LNP site, including Paleozoic (542 to 251 Ma), Mesozoic (251 to 65.5 Ma), and Cenozoic (65.5 Ma to present) tectonic structures. The following SER sections address these regional tectonic features.

Postulated Basement Faults

In FSAR Section 2.5.1.1.4.3.1, the applicant described two postulated basement structures in the site region. These structures include the faults postulated by Applin and Applin (1965) and Barnett (1975). Based on available data, the applicant concluded that these postulated structures are pre-Mesozoic (> 251 Ma) in age and are not capable tectonic features.

Paleozoic Tectonic Structures

In FSAR Section 2.5.1.1.4.3.2, the applicant described four basement structures postulated in the site region, inferred to be Paleozoic in age (> 251 Ma). These structures include the Peninsular arch, the Suwannee-Wiggins suture, the East Suwannee Basin (North Florida Basin), and the Jay fault. The applicant presented information suggesting that the Peninsular arch, a basement high, is spatially associated with a subparallel high in Upper Cretaceous strata that resulted from upwarping produced by compressional tectonics, possibly intermittently during Cenozoic (65.5 Ma to present) time (Miller, 1986). Based on available data, the applicant concluded that these postulated basement structures are not capable tectonic features.

Mesozoic Tectonic Structures

In FSAR Section 2.5.1.1.4.3.3, the applicant described nine basement structures in the site region, inferred from existing published data to be Mesozoic (251 to 65.5 Ma) in age. These structures include the Bahamas and Sunniland fracture zones, Florida Elbow fault, Apalachicola basin, Middle Ground arch, Sarasota arch, South Florida basin, South Georgia rift, and Tampa basin. The applicant documented a Mesozoic age for these structures, and concluded that they are not capable tectonic features.

Cenozoic Tectonic Structures

In FSAR Section 2.5.1.1.4.3.4, the applicant described Cenozoic (65.5 Ma to present) tectonic structures in the site region. These structures include the Brevard, Ocala, and St. Johns platforms; Gulf trough; Jacksonville and Okeechobee basins; Nassau nose; Osceola low; Sanford high; Sarasota arch; Suwannee strait; and faults postulated by Vernon (1951), Carr and Alverson (1959), Pride and others (1966), Sproul and others (1972), Miller (1986), Hutchinson (1992), and Winston (1996). SER Figure 2.5.1-2, reproduced from FSAR Figure 2.5.1-223, shows the locations of the faults, postulated by numerous authors based on apparent displacements inferred from limited outcrops and widely-spaced subsurface borehole data. The applicant stated that the actual existence of many of these faults is controversial and not well-supported by available data, and concluded that neither the faults nor the other structural features are capable tectonic structures.

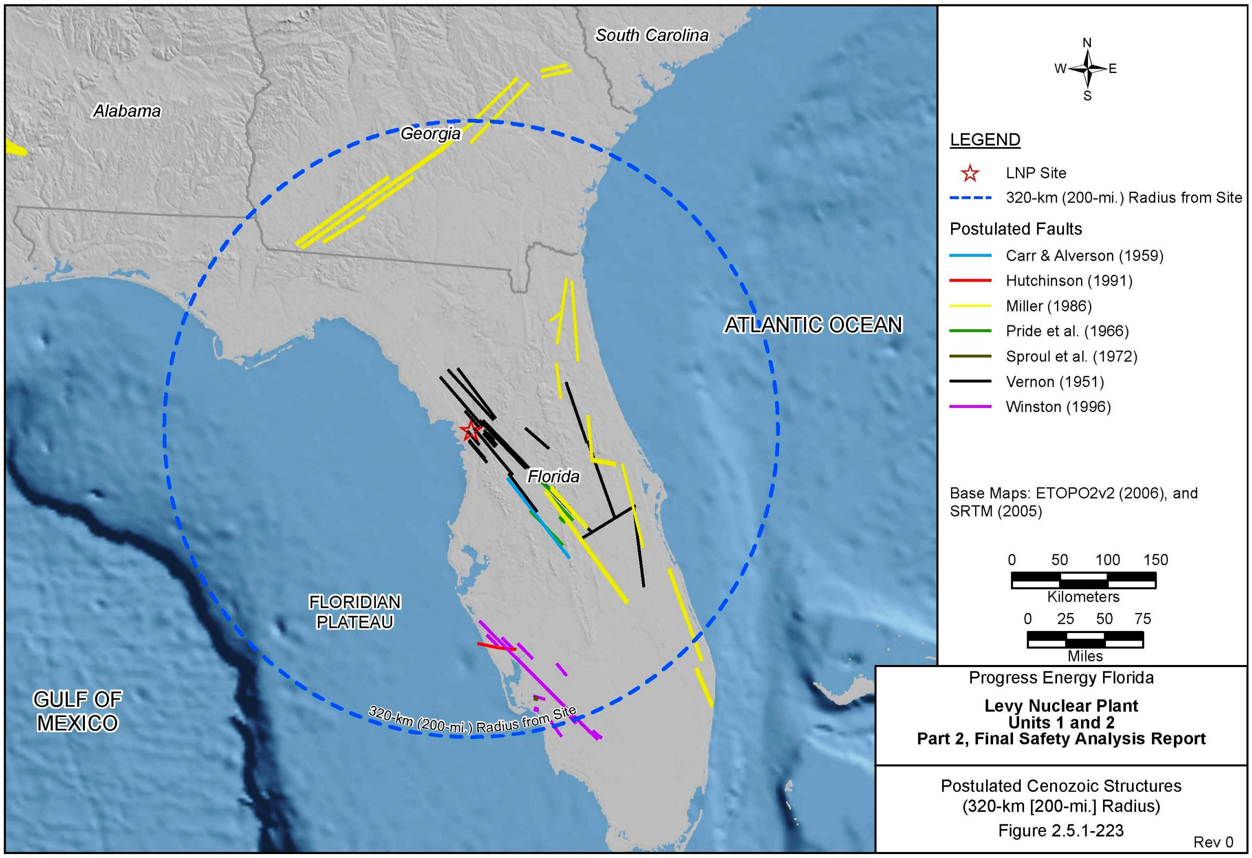


Figure 2.5.1-2. Postulated Cenozoic Tectonic Structures in the LNP Site Region (FSAR Figure 2.5.1-223)

Quaternary Tectonic Structures

In FSAR Section 2.5.1.1.4.3.5, the applicant indicated that there is no geologic or geomorphic evidence of Quaternary faulting in the site region, including the faults postulated by Vernon (1951) to occur within the site area and site vicinity.

2.5.1.2.1.4.4 Significant Seismic Sources at a Distance Greater than 320 km (200 mi)

In FSAR Section 2.5.1.1.4.4, the applicant emphasized the Charleston seismic source zone because, in August 1886, a currently unknown tectonic source in that zone produced one of the largest historical earthquakes in the CEUS in the Charleston, South Carolina area. The applicant incorporated significant new information on source geometry and earthquake recurrence interval for the Charleston earthquake, developed after the initial EPRI studies (EPRI, 1986 and 1989), into an updated Charleston seismic source (UCSS) model that is discussed in detail in FSAR Section 2.5.2. The applicant acknowledged that this model is the same as that used for the Vogtle Electric Generating Plant (VEGP) Early Site Permit (ESP) application (Southern Nuclear Company, 2007), which has been reviewed and approved by NRC staff in NUREG-1923, "Safety Evaluation Report for an Early Site Permit (ESP) at the Vogtle Electric Generating Plant (VEGP) ESP Site." SER Figure 2.5.1-3, reproduced from FSAR Figure 2.5.1-232, illustrates seismicity inside and outside the site region for the time period of 1758 to 2007, including the Charleston region. In addition, the applicant performed sensitivity studies using the CEUS SSC model (NUREG-2115) to address recommendations of the Fukushima Near-Term Task Force described in SECY-12-0025 and evaluate potential seismic hazards at the LNP site in light of these recommendations. SER Section 20.1 presents the staff's evaluation of the sensitivity studies.

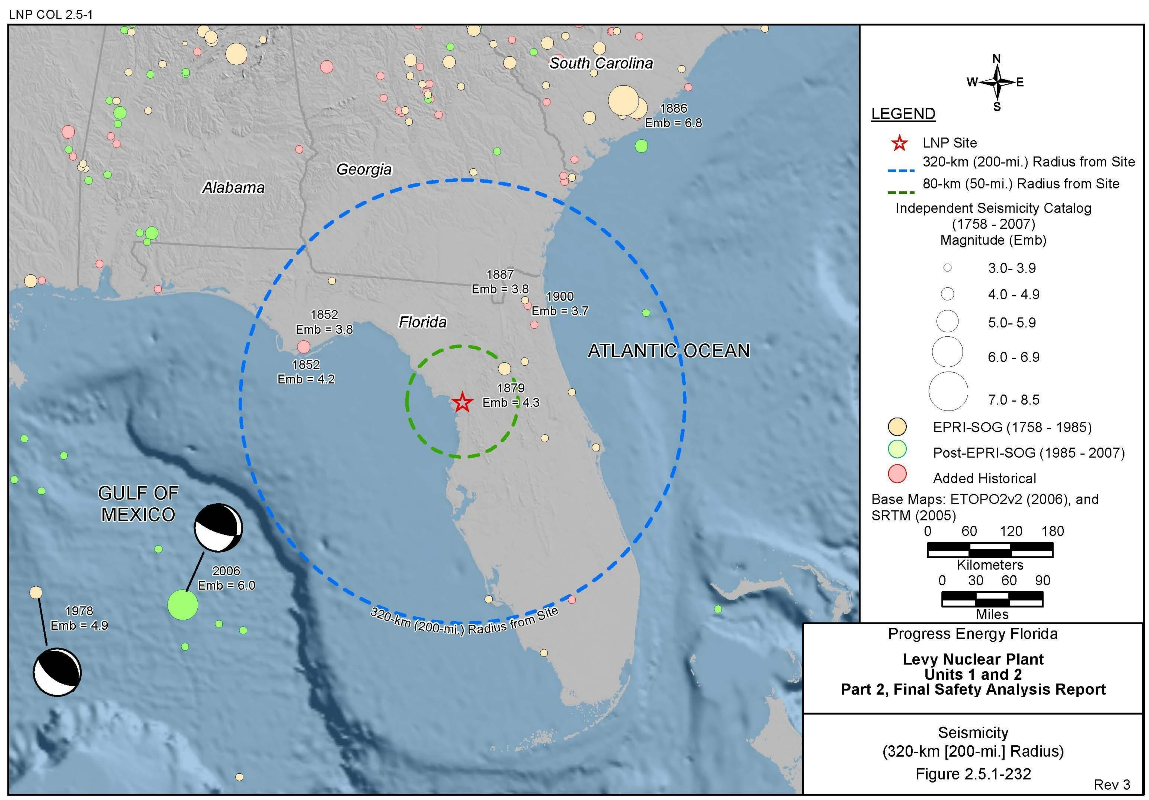


Figure 2.5.1-3. Seismicity in the LNP Site Region and Site Area Between 1758 and 2007. (FSAR Figure 2.5.1–232)

Postulated Associated Tectonic Structures in the Charleston Area

The applicant described five faults postulated to occur in the Charleston area, including the East Coast fault system (ECFS); the Helena Banks fault zone; and the Adams Run, Sawmill Branch, and Summerville faults. The applicant indicated that none of these postulated structures, or any others suggested as occurring in the Charleston area, can be definitively interpreted as a tectonic feature to which the 1886 Charleston earthquake can be related.

Indirect Evidence Related to the Charleston Seismic Source

The applicant discussed the relationship between large global intraplate earthquakes and tectonic environments; liquefaction features produced by the 1886 event and prehistoric earthquakes in the Charleston region; intensity data from the 1886 Charleston earthquake; and instrumental seismicity.

Based on Johnston and others (1994), the applicant documented that the Charleston meizoseismal area (i.e., the area in which an earthquake is most strongly felt) occurs within the region of Mesozoic (251 to 65.5 Ma) or younger extended crust along the southeastern margin of the North American craton, a tectonic environment characterized by large-magnitude earthquakes on a global scale. The applicant also documented that the distribution of liquefaction features produced both by the 1886 Charleston earthquake and pre-1886 events suggest that the Charleston meizoseismal area may encompass the seismic source for 1886 and the pre-1886 events. Intensity data for the 1886 Charleston earthquake also indicate a meizoseismal area centered on Charleston. The applicant further indicated that elevated instrumental seismicity occurs in the Middleton Place-Summer seismic zone, which is located about 20 km (13 mi) northwest of Charleston in the Charleston meizoseismal area. Based on these lines of evidence, the applicant stated that information published since the results of the original EPRI study (EPRI, 1986) strongly indicate that the Charleston seismic source is localized in the meizoseismal area of the 1886 Charleston earthquake, or in the region of coastal South Carolina as constrained by paleoliquefaction data.

M_{max} and Recurrence Interval for the Charleston Seismic Source

In regard to maximum moment magnitude (M_{max}) for the Charleston seismic source, the applicant stated that, given the large uncertainties in working with paleoliquefaction data and the methods for estimating magnitudes from these data, the best representation of M_{max} for the Charleston seismic source should be based on the maximum magnitude of the 1886 earthquake. The applicant reviewed data generated since the original EPRI study (EPRI, 1986), and concluded that M_{max} for the 1886 Charleston earthquake ranges between 6.75 and 7.5.

Concerning recurrence interval for the Charleston seismic source, based on Talwani and Schaeffer (2001), the applicant noted that studies of paleoliquefaction features conducted since the original EPRI study (EPRI, 1986) suggest a recurrence interval for large earthquakes generated by that source of 500-600 years. The applicant incorporated this updated information into the UCSS model as discussed in detail in FSAR Section 2.5.2.

2.5.1.2.1.4.5 Regional Seismicity

In FSAR Section 2.5.1.1.4.5, the applicant indicated that infrequent and low seismicity characterize the U.S. Gulf Coast region in which the LNP site lies (see SER Figure 2.5.1-3). The applicant stated that only 15 earthquakes larger than a body-wave magnitude (m_b) 3.0 have occurred within the LNP site region. The largest event, an 1879 m_b 4.3 earthquake located about 77 km (48 mi) northeast of the LNP site, is the only event within 80 km (50 mi) of the site.

The applicant acknowledged an m_b 6.0 earthquake outside the site region in the Gulf of Mexico, which occurred on 10 September 2006. The focal plane mechanism for that earthquake indicated a compressive stress regime of tectonic origin. On 10 February 2006, an m_b 4.9 event, interpreted to be related to gravity-driven displacement along a growth fault, also occurred outside the site region along the Gulf Coast. The applicant recognized that these two earthquakes may have implications for evaluation of seismicity at the LNP site, and discussed the events in detail in FSAR Section 2.5.2.

2.5.1.2.2 Site Geology

FSAR Section 2.5.1.2 discusses physiography and topography, geomorphology, geologic history, stratigraphy, structural geology, geology, geologic hazard, and engineering geology within the 40 and 8 km (25 and 5 mi) site vicinity and area, respectively. In some of these discussions, the applicant also evaluated the area within the 1 km (0.6 mi) site location. The applicant specifically addressed features commonly developed in karst terrains (e.g., sinkholes) because the LNP site lies within the Limestone Shelf and Hammocks subzone of the Gulf Coastal Lowlands, a geomorphic province underlain by limestones of Eocene age (55.8 to 33.9 Ma), including the Avon Park Formation, which have been subjected to dissolution. The following sections summarize the information the applicant provided in FSAR Section 2.5.1.2.

2.5.1.2.2.1 Site Physiography, Topography, and Geomorphology

FSAR Section 2.5.1.2.1 discusses physiography, topography, and geomorphic provinces within the site vicinity and site area in relation to development of marine terraces and karst terrain, both of which characteristically occur in the site region. The applicant stated that the LNP site lies within the Gulf Coastal Lowlands geomorphic province of the midpeninsular physiographic zone, and that this geomorphic province represents a mature karst terrain overlain by a thin veneer of marine terrace deposits. The other geomorphic province comprising the midpeninsular physiographic zone, the Central Highlands, occurs within the site vicinity as illustrated in SER Figure 2.5.1-4, reproduced from FSAR Figure 2.5.1-234.

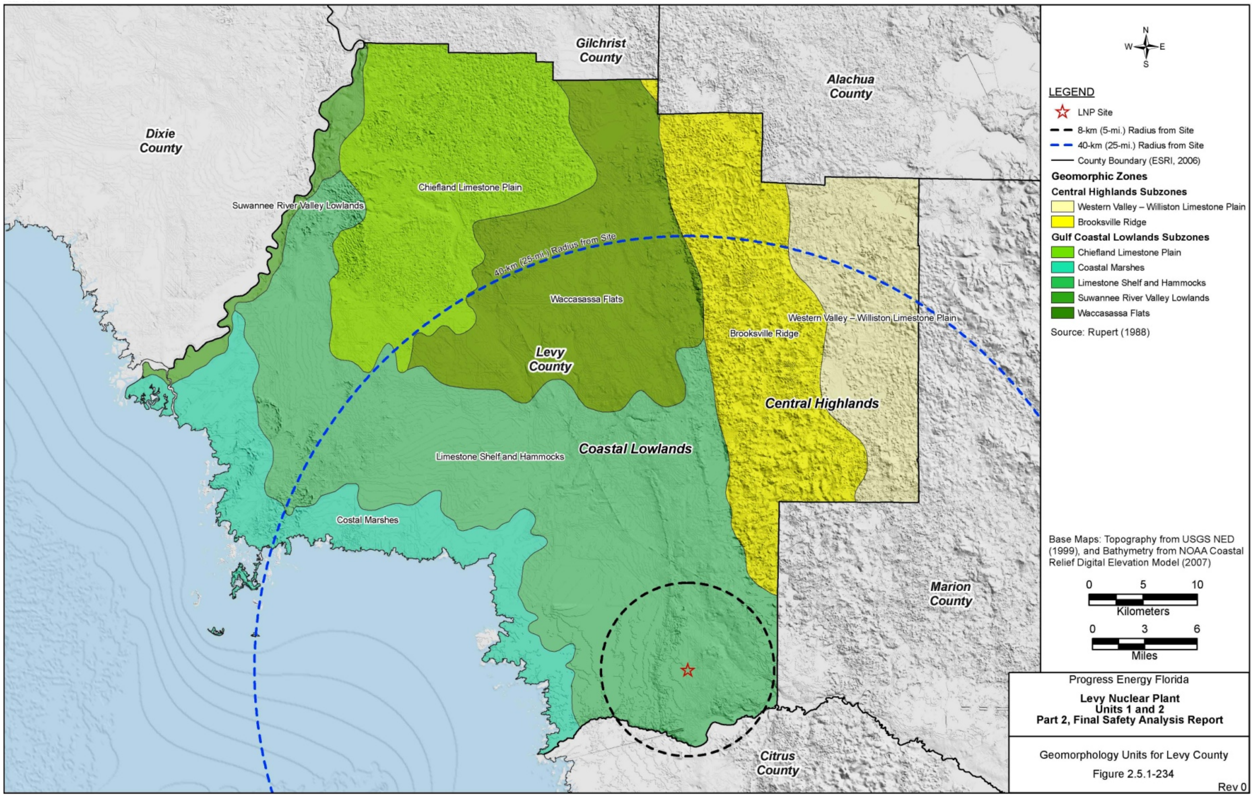


Figure 2.5.1-4. Geomorphic Divisions of Levy County
(FSAR Figure 2.5.1-234)

The applicant noted that the Central Highlands geomorphic province includes a series of highlands and ridges separated by valleys, all of which generally parallel the coastline of the central Florida peninsula. The highlands and ridges, interpreted to be relict coastal features, range in elevation from about 23 to 64 m (75 to 210 ft) amsl. The applicant indicated that the LNP site lies specifically in the Limestone Shelf and Hammocks subzone of the Gulf Coastal Lowlands province (see SER Figure 2.5.1-4), and that this subzone exhibits a highly karstic, irregular, dissolutioned erosional surface composed of Eocene (54.8 to 33.7 Ma) limestones. The karstic limestone units are overlain by sand dunes, ridges, and belts of coastline-parallel paleoshoreline sands associated with the Pamlico marine terrace of Pleistocene (2.6 Ma to 10,000 years) age. The applicant stated that the five marine terraces present in the site vicinity record the long-term effects of late Tertiary (5.3 to 2.6 Ma) to Quaternary (2.6 Ma to present) sea level changes on the stable Florida platform.

2.5.1.2.2.2 Site Vicinity Geologic History

FSAR Section 2.5.1.2.2 summarizes the geologic history of the site vicinity. The applicant indicated that the Florida platform has been tectonically quiescent since Cretaceous (145.5-65.5 Ma) time, allowing a thick sequence of shallow-water marine carbonate rocks to be deposited in the site vicinity, with periodic pulses of clastic sediments interrupting the carbonate deposition. The applicant stated that carbonate deposition ceased on the platform by Middle to Late Pliocene (i.e., between about 3.6-2.6 Ma), due to an influx of clastic sediments, and that total accumulated thickness of sedimentary units in the site vicinity is approximately 1,320 m (4331 ft) based on borehole data. The applicant noted that sea level fluctuations from Miocene (23-5.3 Ma) into Quaternary (2.6 Ma to present) influenced deposition and distribution of sediments on the Florida platform in the site vicinity, and sea level rose to its present-day level following the latest sea level regression during the Pleistocene (2.6 Ma to 10,000 years).

2.5.1.2.2.3 Site Vicinity and Site Area Stratigraphy

FSAR Section 2.5.1.2.3 addresses stratigraphy of the site vicinity and site area. The applicant stated that, within the site vicinity and site area, undifferentiated sediments consisting of surficial sands, clayey sands, and alluvium of Pleistocene (2.6 Ma to 10,000 years) to Holocene (10,000 years to present) age overlie a thick section of Cretaceous (144.5 to 65.5 Ma) and Cenozoic (65.5 Ma to present) carbonates (i.e., limestone and dolomite). The applicant indicated that the Cenozoic carbonate section lies atop basement rocks of Triassic (251 to 201.6 Ma) and Paleozoic (> 251 Ma) age.

The applicant noted that the undifferentiated surficial sediments of Pleistocene to Holocene age are commonly thickest in areas where they accumulated as infilling of karst features. The applicant stated that the surficial sediments mapped at the LNP site generally have a thickness of about 1 to 2 m (3.2 to 6.5 ft). The applicant also noted that sinkholes and related karst features associated with dissolution of the underlying limestone bedrock are common in the site vicinity.

The applicant further indicated that the Avon Park Formation, the foundation unit at the LNP site and the oldest exposed rock unit in Florida, is part of the Cenozoic carbonate section and Middle Eocene (48.6 to 40.4 Ma) in age. The applicant stated that the Avon Park Formation is approximately 243 to 304 m (800 to 1,000 ft) thick in Levy County.

2.5.1.2.2.4 Site Vicinity and Site Area Structural Geology

FSAR Section 2.5.1.2.4 discusses structural geology of the site vicinity and site area. The applicant stated that recent geologic maps encompassing the site vicinity show only a single potential structural feature, the Ocala platform, and no faults. The long axis of the Ocala platform, located about 14 km (8.7 mi) northeast of the LNP site at its nearest point, trends northwest-southeast across midpeninsular Florida. Based on personal communications with regional experts (T. Scott, 2009, and S. Upchurch, 2009), the applicant indicated that the Ocala platform likely resulted from sedimentary, rather than tectonic, processes. The applicant noted that a primary northwest-southeast fracture set parallels the axis of the Ocala platform, while a secondary northeast-southwest fracture set exhibits a strike, which parallels the approximate down dip direction of the flanks of the platform. The applicant recognized that regional fracture systems control stream drainage patterns and sinkhole alignments.

The applicant stated that no known faults occur at the site location based on current field evidence. However, the applicant noted that Vernon (1951) postulated seven northwest-trending faults along the Levy-Citrus County boundary, five of which lie within the LNP site vicinity. The five faults postulated by Vernon (1951) to occur in the site vicinity are as follows:

- Bronson graben – located 24 km (15 mi) northeast of the site.
- Inverness fault – located east of the site within the site area.
- Long Pond fault – located 10 km (6 mi) northeast of the site.
- Unnamed faults "A" and "B" – located 4 km (2.5 mi) southwest and 7 km (4 mi) northeast of the site, respectively.

The applicant documented that subsequent geologic investigations provided no evidence to support the existence of any of the faults proposed by Vernon (1951), and concluded that none of these postulated structures are capable tectonic sources. The applicant also reported two small domal structures, the Homosassa Springs dome located 25 km (15.5 mi) south of the site and the West Levy dome located 45 km (28 mi) northwest of the site. The applicant concluded that these two domal structures pose no geologic hazard for the LNP site because no field evidence exists to indicate that they are tectonically active features.

2.5.1.2.2.5 Site Location Geology

FSAR Section 2.5.1.2.5 discusses geology of the site location, including location-specific geomorphology, stratigraphy, and karst development, based on information derived from field reconnaissance and subsurface exploration. In FSAR Section 2.5.1.2.5.1, the applicant stated that surface morphology is characterized by shallow depressions less than 1 to 2 m (2 to 6 ft) deep above sinkholes or paleosinks, which vary from well-defined, small circular depressions less than 50 m (164 ft) in diameter in the eastern half of the site location to large, irregular depressions up to 600 m (2000 ft) wide in the western half. By analogy with similar morphology of the present-day coastline south of the site in Citrus County, the applicant concluded that this surface morphology indicates older marine terrace surfaces, which have been karstified due to dissolution of carbonate rocks, underlie the site. A thin veneer of Quaternary (2.6 Ma to present) sediments mantle the terrace surfaces.

In FSAR Section 2.5.1.2.5.2, based on results of the geotechnical drilling program conducted at the LNP site to investigate subsurface stratigraphy, the applicant indicated that the Middle Eocene (48.6 to 40.4 Ma) Avon Park Formation is the marine carbonate unit encountered immediately below surficial sedimentary aquifer deposits. The applicant noted that the thickness of Quaternary sediments varied across the site, generally from less than 3 m (10 ft) to about 30 m (100 ft), with an approximate thickness of 2 m (6 ft) beneath the proposed location of the nuclear island and a maximum measured thickness of 73.5 m (241 ft) at one borehole located just beyond the perimeter of the LNP Unit 2 site. The applicant stated that the Avon Park Formation occurs as a soft fossiliferous limestone near the top of the sequence, with increasing dolomitization at depth, particularly in a zone of denser rock at depths around 40 to 60 m (140 to 190 ft). The applicant noted that the Avon Park Formation was softer, and consequently exhibited poorer core recovery, at depths below about 61 m (200 ft).

In FSAR Section 2.5.1.2.5.3, the applicant evaluated the potential for karst development at the site location. The applicant stated that the rectilinear margins of topographic lows, the orientations of depression axes, and the spatial distribution of deeper circular surficial dissolution features suggest control by joint systems in the underlying rock units, including the Avon Park Formation. However, the applicant indicated that the carbonate units in the Avon Park Formation typically exhibit greater degrees of dolomitization than younger limestone units in the site vicinity, and would, therefore, be less susceptible to dissolution and development of karst. The applicant concluded that surface morphology and stratigraphy at the site location are consistent with the anticipated characteristics of a paleokarst landscape mantled by a veneer of Quaternary (2.6 Ma to present) sands. The applicant cross-referenced FSAR Section 2.5.4.1.2.1, and stated that subsurface karst features identified in borings under proposed safety-related structures at the LNP site varied in lateral extent from a few centimeters to about 1.5 m (5 ft) when associated with dissolution controlled by vertical fractures, and from a few centimeters to approximately 3 m (10 ft) in lateral extent when associated with dissolution controlled by horizontal bedding planes.

2.5.1.2.2.6 Site Area Geologic Hazard Evaluation

FSAR Section 2.5.1.2.6 presents an evaluation of potential geologic hazards at the LNP site based on the applicant's review of published information, reconnaissance investigations performed in the site area, discussion with karst experts, and site characterization results. The applicant concluded the following in regard to potential geologic hazards in the site area:

- The site lies in an area of low seismicity and there are no capable tectonic sources in the site area. Therefore, the potential for surface tectonic deformation at the site is minor.
- No natural processes that could cause tectonic uplift are active at the site.
- Unrelieved residual stresses do not pose a hazard to the site.
- Ground failure and differential settlement due to liquefaction do not pose hazards to the site. (The applicant discussed this potential hazard in detail in FSAR Section 2.5.4.)
- Potential surface deformation due to carbonate dissolution and collapse or subsidence related to karst development is the only geologic hazard identified in the LNP site area.

2.5.1.2.2.7 Site Engineering Geology Evaluation

FSAR Section 2.5.1.2.7 addresses the potential engineering significance of geologic and geotechnical features and materials at the site, including zones of alteration, weathering, weakness due to the presence of faults or fault zones, karst, and deformation. In FSAR Section 2.5.1.2.7.1, the applicant cross-referenced FSAR Section 2.5.4 and stated that it addressed engineering behavior of soil and rock materials. In FSAR Section 2.5.1.2.7.2, the applicant indicated that the Avon Park Formation, the bedrock unit underlying the LNP site, has been altered by weathering and dissolution, but no zones of weakness related to faults or fault zones have been identified at the site. Furthermore, the applicant stated that recent studies do not provide evidence of faults postulated by Vernon (1951) to occur in the site vicinity. The applicant acknowledged that smaller-scale fractures and joints parallel to regional fracture trends occur in bedrock outcrops in the site area and in boreholes at the LNP site, and that these discontinuities, particularly in combination with bedding planes along which dissolution may also occur, are key elements controlling the development of karst.

In FSAR Section 2.5.1.2.7.3, the applicant explained that karst features, which occur within the LNP site location, are expected to be associated with vertical fractures and horizontal bedding planes, and that karst-related dissolution and infilled zones, which may exist in the subsurface beneath the LNP foundation, would be addressed through appropriate design considerations as discussed in FSAR Section 2.5.4. In FSAR Section 2.5.1.2.7.4, the applicant stated that, with the exception of possible paleosinkholes, no deformation zones were encountered during site exploration studies for LNP Units 1 and 2, and that excavation mapping would be done during construction to further evaluate the possible existence of deformation zones at the site. Groundwater conditions at the site are discussed in FSAR Sections 2.4 and 2.5.4.6.

2.5.1.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793, “Final Safety Evaluation Report Related to Certification of the AP1000 Standard Design,” and its supplements.

The applicable regulatory requirements for geologic and seismic information are as follows:

- Title 10 of the *Code of Federal Regulations* (10 CFR) 52.79(a)(1)(iii), “Contents of applications; technical information in final safety analysis report,” as it relates to identifying geologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 100.23, “Geologic and seismic siting criteria,” for evaluating the suitability of a proposed site based on consideration of the geologic, geotechnical, geophysical, and seismic characteristics of the proposed site. Geologic and seismic siting factors must include the safe shutdown earthquake (SSE) for the site and the potential for surface tectonic and non-tectonic deformation. The site-specific GMRS satisfies requirements of 10 CFR 100.23 with respect to development of the SSE.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for basic geologic and seismic information are given in Section 2.5.1 of NUREG-0800, “Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants.”

- Regional Geology: In meeting the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23, LNP COL FSAR Section 2.5.1.1 will be considered acceptable if a complete and documented discussion is presented for all geologic (including tectonic and non-tectonic), geotechnical, seismic, and geophysical characteristics, as well as conditions caused by human activities, deemed important for safe siting and design of the plant.

- Site Geology: In meeting the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23, and the guidance in RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants"; Revision 2; RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants"; Revision 2; RG 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites" RG 1.206; and RG 1.208, LNP COL FSAR Section 2.5.1.2 will be considered acceptable if it includes a description and evaluation of geologic (including tectonic and non-tectonic) features, geotechnical characteristics, seismic conditions, and conditions caused by human activities at appropriate levels of detail within areas defined by circles drawn around the site using radii of 40 km (25 mi) for site vicinity, 8 km (5mi) for site area, and 1 km (0.6 mi) for site location.

In addition, the geologic characteristics should be consistent with appropriate sections from RG 1.132, Revision 2; RG 1.138, Revision 2; RG 1.198; RG 1.206; and RG 1.208.

2.5.1.4 *Technical Evaluation*

The NRC staff reviewed Section 2.5.1 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of information presented in the FSAR and the DCD completely represents the required information related to basic geologic and seismic characteristics. The staff's review confirmed that information contained in the application or incorporated by reference addresses the information required for this review topic. NUREG-1793 and its supplements document the results of the staff's evaluation of the information incorporated by reference into the LNP COL application.

The staff reviewed the following information in the LNP COL FSAR.

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.5-1

The NRC staff reviewed LNP COL 2.5-1 regarding the geologic, seismic, and geophysical information included in Section 2.5.1 of the LNP COL FSAR. The COL information item in Section 2.5.1 of the AP1000 DCD states:

> Combined License applicants referencing the AP1000 certified design will address the following regional and site-specific geological, seismological, and geophysical information as well as conditions caused by human activities: (1) structural geology of the site, (2) seismicity of the site, (3) geological history,

(4) evidence of paleoseismicity, (5) site stratigraphy and lithology, (6) engineering significance of geological features, (7) site groundwater conditions, (8) dynamic behavior during prior earthquakes, (9) zones of alteration, irregular weathering, or structural weakness, (10) unrelieved residual stresses in bedrock, (11) materials that could be unstable because of mineralogy or physical properties, and (12) effect of human activities in the area.

Based on the discussion of the basic geologic and seismic information presented in LNP COL FSAR Section 2.5.1, the staff concludes that the applicant provided the information required to satisfy LNP COL 2.5-1.

The technical information presented in LNP COL FSAR Section 2.5.1 resulted from the applicant's review of existing geologic and seismicity data and published literature cited by the applicant; discussions with individuals who have conducted recent research in and around the site area; field reconnaissance studies in the site vicinity and site area and at the site location; lineament analyses using aerial photographs and remote sensing imagery; and detailed investigations performed for the LNP site, including subsurface borings, surface geophysical testing, and downhole geophysical logging and seismic testing. The applicant also provided limited information applicable to the LNP site as derived from the FSAR prepared by Florida Power Corporation (Florida Power Corporation, 1976) for the CR3, which is located about 18 km (11 mi) southwest of the LNP COL site. Through the review of LNP COL FSAR Section 2.5.1, the staff determined whether the applicant had complied with the applicable regulations and conducted the investigations at an appropriate level of detail in accordance with RG 1.208.

LNP COL FSAR Section 2.5.1 includes geologic and seismic information the applicant collected in support of the vibratory ground motion analysis and the site-specific GMRS provided in FSAR Section 2.5.2. RG 1.208 recommends that applicants update the geologic, seismic, and geophysical database and evaluate any new data to determine whether revisions to the existing seismic source models are necessary. Consequently, the staff focused the review on geologic and seismic data published since the mid-to-late 1980s to assess whether these data indicate a need to update the existing seismic source models.

The staff visited the site in April 2009 (ML092600064), supported by technical experts from the U.S. Geological Survey (USGS), and interacted with the applicant and its consultants in regard to the geologic, seismic, geophysical, and geotechnical investigations being conducted for the LNP COL application. During this site visit, the staff examined core samples from the initial site characterization boreholes placed at the locations of containment structures and turbine buildings for LNP Units 1 and 2, as well as exposures of the Avon Park Formation along the Waccasassa River about 25 km (16 mi) northwest of the site. The core samples allowed staff to examine subsurface stratigraphy at the site, and the outcrops along the river permitted staff to observe and measure spacing and orientation of fractures in the Avon Park Formation. The staff also visited the site in September 2009 to examine core samples from the test grouting program. The staff noted grout uptake in a single vertical fracture intersected by one of the grout boreholes. Also during the September 2009 site audit, the staff examined exposures of the Avon Park Formation at the abandoned Gulf Hammock quarry about 19 km (12 mi) north-northwest of the LNP site, which again permitted staff to observe and measure spacing and orientation of fractures in the Avon Park Formation. In addition, in February 2010 at the applicant's records facility in Virginia, the staff examined boring logs, core photographs, and written core descriptions for 6 additional boreholes, located to be offset approximately 1.5 m (5 ft) from the position of the initial site characterization boreholes. These "offset" boreholes were drilled using controlled coring techniques to improve core recovery and further characterize soft zones postulated to mark horizons of low recovery in the initial site characterization boreholes for LNP Units 1 and 2. The two site visits and the examination of boring logs, core photographs, and core descriptions enabled the staff to assess and confirm the interpretations, assumptions, and conclusions the applicant made regarding the basic geologic and seismic information for the LNP site, including features related to karst development.

The following SER Sections 2.5.1.4.1, "Regional Geology," and 2.5.1.4.2, "Site Geology," present the staff's evaluation of the information the applicant provided in LNP COL FSAR Section 2.5.1 and the applicant's responses to RAIs for that FSAR section. In addition to the RAIs addressing specific technical issues related to regional and site geology of the LNP site, discussed in detail below, the staff also prepared several editorial RAIs to further clarify certain descriptive statements the applicant made in the FSAR and to qualify geologic features illustrated in FSAR figures. These editorial RAIs are not discussed in this technical evaluation. Also, RAIs related to geologic issues resolved in FSARs previously prepared for other sites in the CEUS are not discussed in detail in this technical evaluation for the LNP site, but rather addressed by cross-reference to and a summary of the pertinent information used to satisfactorily resolve the issues as presented in those FSARs.

2.5.1.4.1 Regional Geology

The staff focused the review of LNP COL FSAR Section 2.5.1.1 on the descriptions the applicant provided for physiography, topography, geologic history, stratigraphy, tectonic setting, and seismicity within the 320-km (200-mi) radius LNP site region. The staff also focused on the description of significant seismic sources outside the site region the applicant provided under the discussion of regional tectonic setting.

2.5.1.4.1.1 Regional Physiography and Topography

In FSAR Section 2.5.1.1.1, the applicant described the physiography and topography of the Coastal Plain physiographic province in the site region, including the Sea Island, East Gulf, and Floridian sections of that physiographic province. SER Figure 2.5.1-1 shows the location of the LNP site and its spatial relationship to these three sections of the Coastal Plain physiographic province. The LNP site lies within the Floridian section of the Coastal Plain province.

The staff focused the review of FSAR Section 2.5.1.1.1 on the applicant's discussion of the characteristics of rock units within the Coastal Plain physiographic province and the mechanism for and timing of the differential emergence of the Floridian Coastal Plain section of the Coastal Plain physiographic province in which the site lies. In RAI 2.5.1-13, the staff asked the applicant to clarify the use of the adjective "weak" when describing the limestones contained in the East Gulf Coastal Plain section of the site region. In response to RAI 2.5.1-13, the applicant stated that “weak” refers to these limestones being less resistant to erosion, without any implication related to mechanical strength of the rock unit, while “stronger” indicates a rock unit that is more resistant to erosion (e.g., sandstones). The applicant incorporated changes in LNP COL FSAR Section 2.5.1.1.1.1.2 to replace the adjective “weak” with the phrase “more easily eroded” when referring to limestones and shales, and “less easily eroded” when discussing sandstones.

Based on review of the response to RAI 2.5.1-13 and LNP COL FSAR Section 2.5.1.1.1.1.2, the staff concludes that the applicant adequately clarified the descriptive term “weak” as applied to the limestone units, which occur in the East Gulf Coastal Plain section of the site region. The staff makes this conclusion because the applicant clearly explained that “weak” refers to limestone and shale units that are less resistant to erosion due to its physical properties, rather than to any mechanical weakness that could pose a potential problem for stability of the foundation rock units at the LNP site. Consequently, the staff considers RAI 2.5.1-13 to be resolved.

In RAI 2.5.1-14, the staff asked the applicant to discuss the mechanism for and timing of the differential emergence of the Floridian Coastal Plain section of the Coastal Plain physiographic province in which the site lies in order to document that this emergence is not the result of Cenozoic (65.5 Ma to present) tectonic deformation. In response to RAI 2.5.1-14, the applicant summarized information from published literature cited by the applicant documenting that the observed elevation differences are the result of depositional and erosional processes primarily associated with sea level fluctuations, and that no evidence exists to suggest Cenozoic tectonic deformation as the causative mechanism. Based on robust data presented by Willett (2006), the applicant also documented calculations that show karst areas in Florida are losing about 1 m (3 ft) of limestone every 160,000 years due to dissolution, resulting in isostatic uplift of the Florida carbonate platform of as much as 58 m (190 ft) since early Quaternary time (i.e., < 2.6 Ma). The applicant further reported that Means (2009) suggested lithospheric flexure due to sediment loading as another non-tectonic uplift mechanism.

Based on review of the response to RAI 2.5.1-14, and independent review of published geologic information cited by the applicant, the staff concludes that the applicant documented that non-tectonic processes related to erosion, isostatic adjustment, and sea level fluctuations produced the differential emergence of the Floridian Coastal Plain section of the Coastal Plain physiographic province in which the site lies. Based on information derived from Willett (2006) and Means (2009), the staff further concludes that there is no evidence for Cenozoic tectonic deformation in the site area, and that the likelihood of neotectonic (i.e., < 5.3 Ma in age) deformation in the site region is negligible. The staff draws these conclusions because a preponderance of data collected by experts on geologic evolution of the site region strongly supports non-tectonic processes as the causative mechanism for emergence of the Florida Coastal Plain section. Consequently, the staff considers RAI 2.5.1-14 to be resolved.

Based on the review of LNP COL FSAR Section 2.5.1.1.1 and the responses to RAIs 2.5.1-13 and 2.5.1-14, the staff finds that the applicant provided a thorough and accurate description of regional physiography and topography in support of the LNP COL application.

2.5.1.4.1.2 Regional Geologic History

In FSAR Sections 2.5.1.1.2.1 through 2.5.1.1.2.3, the applicant discussed Late Proterozoic (> 542 Ma), Paleozoic (542 to 251 Ma), Mesozoic (251 to 65.5 Ma), and Cenozoic (65.5 Ma to present) geologic history of the LNP site region, including the Florida platform on which the site is located, concentrating on tectonic evolution and depositional history of sedimentary rock units for the site region and the platform. The applicant documented that tectonic deformation in the site region occurred mainly in pre-Cretaceous (> 65.5 Ma) time; that the Florida platform represents long-term sedimentation in a tectonically stable area as evidenced by undisturbed Upper Cretaceous (99.6 to 65.5 Ma) and Tertiary (65.5 to 2.6 Ma) strata on the platform; and that late Quaternary (< 2.6 Ma to present) deposits in the Florida Keys do not record significant uplift, subsidence, or tectonic deformation of the platform.

The staff focused the review of FSAR Section 2.5.1.1.2 on the Cenozoic depositional history of the Florida platform to ensure that no sedimentation patterns reflected Quaternary tectonic deformation in the site region. Based on independent review of the data sources the applicant provided, the staff concludes that tectonic deformation in the site region occurred mainly in pre-Cretaceous time because no existing data indicate younger Cenozoic (65.5 Ma to present) tectonic deformation. The staff further concludes that the Florida platform represents long-term sedimentation in a tectonically stable area since undisturbed Upper Cretaceous (99.6 to 65.5 Ma) and Tertiary (65.5 to 2.6 Ma) strata occur on the platform and no evidence exists for late Quaternary deformation.

Based on review of the LNP COL FSAR Section 2.5.1.1.2, the staff finds that the applicant provided a thorough and accurate description of the regional geologic history in support of the LNP COL application.

2.5.1.4.1.3 Regional Stratigraphy

In FSAR Section 2.5.1.1.3, the applicant described stratigraphic relationships for pre-Cretaceous (> 145.5 Ma), Cretaceous (145.5 to 65.5 Ma), and post-Cretaceous (< 65.5 Ma) rock units, which occur in the LNP site region. The applicant specifically addressed the foundation unit for LNP Units 1 and 2, the Middle Eocene (48.6 to 40.4 Ma) Avon Park Formation.

The staff focused the review of FSAR Section 2.5.1.1.3 on the applicant's descriptions of the Avon Park Formation. In RAI 2.5.1-4, the staff asked the applicant to describe the composition, thickness, lateral distribution, and material properties of a "shelf" occurring within the Avon Park Formation, as defined by low shear wave velocity (V_S) values. In response to RAI 2.5.1-4, the applicant stated that the "shelf" is a dolomitized stratigraphic horizon within the Avon Park Formation. The applicant indicated that this horizon exhibits little to no dip, and appears to underlie and extend laterally beyond the footprint of LNP Units 1 and 2. The applicant provided figures locating the dolomitized "shelf" horizon in relation to LNP Units 1 and 2, as well as tables summarizing the physical properties of this dolomitized horizon.

Based on its review of FSAR Section 2.5.1.1.3 and the applicant's response to RAI 2.5.1-14, the staff concludes that the applicant adequately described the stratigraphic "shelf" horizon within the Avon Park Formation, which underlies LNP Units 1 and 2. The staff makes this conclusion because the information provided by the applicant characterized this stratigraphic horizon in regard to its composition, thickness, lateral extent, material properties, and engineering parameters. Consequently, the staff considers RAI 2.5.1-4 to be resolved.

Based on review of LNP COL FSAR Section 2.5.1.1.3 and the applicant's response to RAI 2.5.1-4, the staff finds that the applicant provided a thorough and accurate description of the regional stratigraphy in support of the LNP COL application.

2.5.1.4.1.4 Regional Tectonic Setting

FSAR Section 2.5.1.1.4 discusses the tectonic setting of the site region. The applicant described the regional tectonic setting in terms of contemporary tectonic stress; structural setting and geophysical framework; tectonic features within a 320-km (200-mi) radius of the site; and significant seismic sources at a distance greater than 320 km (200 mi) from the LNP site. The staff focused the review of LNP COL FSAR Section 2.5.1.1.4 on the discussion of postulated tectonic features in the site region and possible significant seismic sources outside the site region, including the Charleston seismic source zone.

2.5.1.4.1.4.1 Tectonic Features in the Site Region

In RAI 2.5.1-17, the staff asked the applicant to discuss the data used by Barnett (1975) that postulated a basement fault passing through or near the site location, as suggested by FSAR Figure 2.5.1-222. In response to RAI 2.5.1-17, the applicant stated that Barnett (1975) did not provide detailed descriptions or justification for the locations of most of the basement faults he postulated, including the fault shown on FSAR Figure 2.5.1-222, which he inferred displaced pre-Middle Jurassic (> 161 Ma) basement rocks in the LNP site area. The applicant noted that, due to the scale of the maps presented by Barnett (1975), it was not possible to determine the exact location of the postulated basement structure relative to the site. The applicant indicated that Barnett (1975) based his interpretations of basement faulting on data from about eighty widely-spaced and sparsely-distributed wells that penetrated the basement, as well as well logs and geophysical and geologic data derived from published literature sources cited by the applicant. The applicant stated that the data cited by Barnett (1975) do not require a significant offset in the top of basement, as would be expected if a normal fault of large displacement existed, and that the structures postulated by Barnett (1975) are not expressed in gravity or magnetic maps for the site vicinity. Based on the fact the no data show anomalies to suggest faulting in the LNP site vicinity, the applicant concluded that no definitive evidence exists for faulting there.

Based on the fact that no current data suggest the presence of post-middle Jurassic faulting in the site vicinity, the staff concludes that the applicant provided sufficient information in the response to RAI 2.5.1-17 to document the speculative nature of the basement faults postulated by Barnett (1975), and that, if these basement structures exist, there is no evidence to demonstrate post-Middle Jurassic activity associated with the structures in the site vicinity. Consequently, the staff considers RAI 2.5.1-17 to be resolved.

In RAI 2.5.1-18, the staff asked the applicant to locate all regional tectonic structures discussed in FSAR Section 2.5.1.1.4.3, but which were not shown in referenced FSAR Figures 2.5.1-208 and 2.5.1-209, to enable a thorough assessment of tectonic features found in the LNP site region in regard to whether they may represent capable tectonic structures. In the response to RAI 2.5.1-18, the applicant incorporated changes to FSAR Section 2.5.1.1.4.3, including modifications to Figures 2.5.1-209 and 2.5.1-222, to further qualify the locations, ages, and types of deformation for tectonic structures in the site region.

Based on review of the applicant's response to RAI 2.5.1-18 and modifications implemented for figures and text in Revision 4 of LNP COL FSAR Section 2.5.1.1.4.3, the staff concludes that the applicant provided appropriate changes in Revision 4 of FSAR Section 2.5.1.1.4.3. The staff makes this conclusion because the modifications provided in Revision 4 of FSAR Section 2.5.1.1.4.3 locate all regional tectonic structures that lie within the LNP site region and qualify the ages and styles of deformation for these structures. Consequently, the staff considers RAI 2.5.1-18 resolved.

2.5.1.4.1.4.2 Charleston Area Tectonic Features

In RAI 2.5.1-21, the staff asked the applicant to summarize existing information on the following tectonic features postulated to occur in the Charleston area: the Ashley River, Charleston, Cooke, Drayton, Gants, and Woodstock faults. FSAR Figure 2.5.1-225 and Table 2.5.1-201 include these faults, but they are not discussed in detail in FSAR Section 2.5.1.1.4.4. In response to RAI 2.5.1-21, the applicant proposed changes to FSAR Section 2.5.1.1.4.4 and incorporated those changes in Revision 4 of LNP COL FSAR Section 2.5.1.1.4.4 to provide a discussion of the six tectonic features in the Charleston area included in FSAR Figure 2.5.1-225 and Table 2.5.1-201, but not initially discussed in the FSAR.

Based on review of the applicant's response to RAI 2.5.1-21 and the modifications included in Revision 4 of FSAR Section 2.5.1.1.4.4, the staff concludes that the applicant provided appropriate changes in Revision 4 of FSAR Section 2.5.1.1.4.4 because the modifications present a discussion of all tectonic features in the Charleston area. Consequently, the staff considers RAI 2.5.1-21 to be resolved.

In RAI 2.5.1-22, the staff asked the applicant to summarize the basis for the conclusion, presented in FSAR Section 2.5.1.1.4.4, that there is low confidence that the ECFS exists. In response to RAI 2.5.1-22, the applicant discussed several studies that assessed the ECFS as a potential seismic source, including the study for the North Anna ESP application as summarized in NUREG-1835 ("Safety Evaluation Report for an Early Site Permit (ESP) at the North Anna ESP Site"). In NUREG-1835, the NRC staff concluded that the geologic, seismic, and geomorphic evidence for the ECFS-North presented by Marple and Talwani (2000) is uncertain, and that most data apply to the southern and central segments of the ECFS. The applicant also pointed out that the VEGP ESP application (SNC, 2007) indicates that the ECFS-South segment is included in the Charleston area seismic source zone and, therefore, need not be incorporated as a separate and distinct seismic source for the LNP site.

Based on the detailed assessment of the ECFS for the North Anna ESP application as discussed in NUREG-1835 and as cited by the applicant in the response to RAI 2.5.1-22, the staff concludes that there is low confidence in the existence of the postulated northern and central segments of the ECFS. The staff further concludes that the updated Charleston seismic source model the applicant used incorporates the southern segment of the ECFS, which lies closest to the LNP site. Consequently, the staff considers RAI 2.5.1-22 to be resolved.

In RAI 2.5.1-45, the staff asked the applicant to discuss the potential tectonic significance of features in the vicinity of the Charleston seismic source, as shown in FSAR Figure 2.5.1-228, which Weems and Lewis (2002) interpreted to exhibit relative uplift during the last 34 Ma (i.e., possibly during Quaternary time). In response to RAI 2.5.1-45, the applicant stated that Weems and Lewis (2002) acknowledged that the areas shown in FSAR Figure 2.5.1-228, which they interpreted to possibly show uplift over the past 34 Ma based mainly on the irregular paleo-topographic surface shown by the bases of Oligocene (33.9 to 23 Ma) through Pliocene (5.3 to 2.6 Ma) units, could be explained either by buried erosional surfaces, syn-depositional or post-depositional tectonic warping, or a combination of those two factors. Based on examination of structure contour maps presented by Weems and Lewis (2002) drawn on the bases of the Oligocene through Pliocene units, the applicant concluded that uplift and subsidence patterns do not persist through time in the same locations, and that the intervening structural lows between the proposed uplifts are highly suggestive of erosion along ancient river channels. This conclusion drawn by the applicant agrees with that made by Southern Nuclear Company in its update of the Charleston seismic source for the VEGP site (SNC, 2006).

Based on the applicant's response to RAI 2.5.1-45 and the staff's independent review of the information presented by Weems and Lewis (2002), the staff concludes that any uplift that may have occurred in the vicinity of Charleston, as proposed by Weems and Lewis (2002) during the last 34 Ma, if it occurred, was pre-Quaternary (< 2.6 Ma) in age. The staff draws this conclusion because Weems and Lewis (2002) documented that the paleo-topographic relief observed at the base of one Oligocene formation in this vicinity could not have formed as a result of post-Oligocene (< 23 Ma) tectonic deformation based on the moderate dip and lack of topographic relief on an overlying unit of Upper Oligocene (28.4 to 23 Ma) age. This field relationship strongly suggests that no post-Oligocene tectonic uplift or subsidence occurred. Consequently, the staff considers RAI 2.5.1-45 to be resolved.

2.5.1.4.1.4.3 Earthquakes in Areas of Extended Crust

In RAI 2.5.1-24, the staff asked the applicant to discuss the potential for large-magnitude earthquakes in areas of extended continental crust, which includes the site region, based on interpretations presented in the current literature cited by the applicant. In response to RAI 2.5.1-24, the applicant indicated that Johnston and others (1994) used a global catalog of moderate to large historical seismicity from SCRs to determine that the largest SCR earthquakes (**M** > 7) occurred in areas of extended crust. The applicant noted that Johnston and others (1994) determined a mean magnitude of **M** 6.3 with a standard deviation of 0.5 for areas of non-extended crust, and a mean magnitude of **M** 6.4 with a standard deviation of 0.84 for extended crust. The applicant also reported that Schulte and Mooney (2005) presented an updated global earthquake catalog, which included **M** 4.5 or larger events for SCRs, and re-evaluated the correlation of intraplate seismicity with ancient extensional rifts, and that their study demonstrated that 52 percent of all seismic events occurred within extended crust. Based on limited borehole data, the applicant noted that crust in the LNP site region experienced some extension during the Mesozoic (251 to 65.5 Ma), although the total amount of crustal extension was minimal. The applicant confirmed that the maximum magnitude distribution for seismic sources in the LNP site region used in the updated seismic source model, discussed in detail in FSAR Section 2.5.2, captures an approximate range of **M** 4.5 to 7.7, such that the PSHA characterization for the LNP site allows for the possible occurrence of large earthquakes in the site region.

Based on the applicant's response to RAI 2.5.1-24, and an independent review of published information cited by the applicant related to large-magnitude earthquakes in areas of extended continental crust, the staff concludes that the applicant analyzed current data to assess the potential for large earthquakes in areas of extended crust, including the site region, and documented that the PSHA characterization for the LNP site properly allows for the possible occurrence of large earthquakes in the site region due to the magnitude range captured in the PSHA. The staff makes this conclusion because interpretations from the current literature cited by the applicant related to maximum magnitude of earthquakes that may occur in areas of extended continental crust, which the staff independently reviewed, support the applicant's statement that the PSHA for the LNP site allows for the occurrence of large earthquakes in the site region. Consequently, the staff considers RAI 2.5.1-24 to be resolved.

2.5.1.4.1.4.4 Staff Conclusions on Regional Tectonic Setting

Based on its review of LNP COL FSAR Section 2.5.1.1.4, the applicant's responses to RAIs 2.5.1-17, 2.5.1-18, 2.5.1-21, 2.5.1-22, 2.5.1-24, and 2.5.1-45, and changes incorporated in Revision 4 of FSAR Section 2.5.1.1.4, the staff finds that the applicant provided thorough and accurate descriptions of the regional tectonic setting of the LNP site, including contemporary tectonic stress, regional structural setting and geophysical framework, regional tectonic structures within a 320-km (200-mi) radius of the site, significant seismic sources at a distance greater than 320 km (200 mi) from the site, and regional seismicity. The staff also concludes that the descriptions provided in LNP COL FSAR Section 2.5.1.1.4 reflect the current literature cited by the applicant and state of knowledge and meet the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23.

2.5.1.4.2 Site Geology

NRC staff focused the review of LNP COL FSAR Section 2.5.1.2, "Site Geology," on the descriptions the applicant provided for physiography, topography, geomorphology, geologic history, stratigraphy, structural geology, geologic hazard and engineering geology within the 40 and 8 km (25 and 5 mi) LNP site vicinity and area, respectively. The staff also focused on the descriptions the applicant provided for certain of these topics for the area within 1 km (0.6 mi) of the site (i.e., the site location). The staff concentrated specifically on the applicant's descriptions of the geologic characteristics, which may enhance the development of karst, including regional, site vicinity, site area, and site location fracture patterns, and of the evidence that the site vicinity has been tectonically quiescent since the beginning of Cretaceous time (i.e., 145.5 Ma).

2.5.1.4.2.1 Site Physiography, Topography, and Geomorphology

In LNP COL FSAR Section 2.5.1.2.1, the applicant described physiography, topography and geomorphology of the LNP site vicinity and site area. The applicant stated that the LNP site lies within the Gulf Coastal Lowlands geomorphic province of the midpeninsular physiographic zone, and that this geomorphic province represents an old karst terrain overlain by marine terrace sediments deposited on a tectonically stable Florida platform during previous higher sea level stands.

Based on review of LNP COL FSAR Section 2.5.1.2.1, as well as independent review of current literature cited by the applicant on physiography, topography, and geomorphology of the site vicinity and site area, the staff finds that the applicant provided a complete and accurate description of site physiography, topography, and geomorphology in support of the LNP COL application

2.5.1.4.2.2 Site Vicinity Geologic History

FSAR Section 2.5.1.2.2 summarizes geologic history of the Florida platform, which includes the site vicinity, from Late Proterozoic (> 542 Ma) to the present. The applicant stated that the Florida Platform has been tectonically quiescent from Cretaceous (145.5 to 65.5 Ma) into Holocene (10,000 years to present) time. The applicant noted that sea level fluctuations, rather than tectonic events, affected sediment distribution in the Florida platform throughout the Neogene (23 to 2.6 Ma) and Quaternary (2.6 Ma to present), and that sea level rose to its present level from Pleistocene (2.6 Ma to 10,000 years) to the present time.

Based on review of LNP COL FSAR Section 2.5.1.2.2, as well as independent review of current literature cited by the applicant on the geologic and tectonic setting of the Florida platform, which documented that the site vicinity has been tectonically quiescent since the start of Cretaceous time, the staff finds that the applicant provided a complete and accurate description of site vicinity geologic history in support of the LNP COL application.

2.5.1.4.2.3 Site Vicinity and Site Area Stratigraphy

FSAR Section 2.5.1.2.3 describes the stratigraphy of the site vicinity and site area. The applicant stated that the lowermost and oldest stratigraphic units are Paleozoic (542 to 251 Ma) shales and quartzite sands overlain by Triassic (252 to 201.6 Ma) diabase. Cretaceous (145.5 to 65.5 Ma) and Cenozoic (65.5 Ma to present) carbonates, consisting of both limestone and dolomite overlain by undifferentiated sediments (surficial sands, clayey sands, and alluvium) of Pleistocene (2.6 Ma to 10,000 yr) to Holocene age (10,000 years to present), comprise the uppermost stratigraphic units. The staff focused the review of FSAR Section 2.5.1.2.3 on aspects of the stratigraphy that may be indicative of karst in the site vicinity and site area.

2.5.1.4.2.3.1 Surficial Quaternary Deposits

In RAI 2.5.1-8, the staff asked the applicant to evaluate the possibility that aerial distribution of thicker surficial Quaternary deposits in areas of lower surface topography may reflect local collapse above dissolution cavities at depth, which allowed deposition of thicker surficial deposits. From cross sections based on borehole data, illustrated in FSAR Figures 2.5.4.2-203A and 2.5.4.2-202A, thickness of Quaternary (2.6 Ma to present) sediments varies from less than 3 m (10 ft) in the site area to at least 24 m (80 ft) at locations near LNP Units 1 and 2. In response to RAI 2.5.1-8, the applicant stated that erosional episodes related to sea level fluctuations removed sediment from the Ocala platform, eventually exposing the Upper Eocene (about 40.4-33.9 Ma) Avon Park Formation, upon which Quaternary sediments accumulated to variable thicknesses. The applicant noted that the erosional surface atop the carbonate sediments in the LNP site region includes incised paleochannels filled with Quaternary sediments, some of which exhibit up to 30 m (98 ft) of relief. Because of the scarcity of dissolution voids encountered in the LNP site borings and the documented erosional and depositional history of the site vicinity, the applicant concluded that the most plausible interpretation of the increased thickness of Quaternary sediments observed in the borings is deposition in paleochannels. As part of the response to RAI 2.5.1-8, the applicant proposed changes to FSAR Sections 2.5.1.2.1.1, 2.5.1.2.1.3, 2.5.1.2.3.3, 2.5.1.2.3.6, 2.5.1.2.5.2, and 2.5.1.2.5.3 to further clarify information regarding LNP site stratigraphy. The staff finds these changes acceptable and verified that the applicant incorporated the changes in LNP COL FSAR Section 2.5.1.2.

Based on review of the applicant’s response to RAI 2.5.1-8 and the changes implemented in FSAR Section 2.5.1.2, the staff concludes that sediment-filled paleochannels are an acceptable explanation for the observed thick Quaternary sediments in the LNP borings. The staff draws this conclusion because there is evidence for this mode of sediment accumulation in the site region, and site characterization boreholes revealed only a few small subsurface voids. In addition, during the site audit conducted in September 2009, the staff confirmed that there is a paucity of subsurface dissolution cavities at the LNP site based on grout uptake in the slanted boreholes drilled for the grout testing program. In February 2010, the staff also examined boring logs, core photographs, and written core descriptions for the six “offset” boreholes, drilled using controlled coring techniques to improve core recovery, which documented that the low recovery horizons noted in the initial site characterization boreholes for LNP Units 1 and 2 marked soft zones in the normal stratigraphic sequence, rather than large subsurface dissolution voids. Consequently, the staff considers RAI 2.5.1-8 to be resolved.

In RAI 2.5.1-9, the staff asked the applicant to discuss whether reactivity to hydrochloric acid (HCL) was the sole test performed to differentiate unconsolidated Quaternary deposits from calcareous silts derived from weathered Avon Park limestone in site characterization boreholes drilled at LNP Units 1 and 2. In response to RAI 2.5.1-9, the applicant stated that Quaternary clastic sediments at the LNP site consist mainly of well-sorted fine quartz sands and silty sands with interbedded clays, and show little reaction to HCL due to a lack of carbonate. The applicant stated that weathered Avon Park Formation carbonates typically lack clastic materials.

Based on review of the applicant's response to RAI 2.5.1-9, the staff concludes that the applicant clarified the additional criterion used to distinguish unconsolidated Quaternary deposits from underlying weathered Avon Park Formation limestone to enable a reasonable estimate of the thickness of Quaternary deposits at the LNP site. The staff makes this conclusion because the observed variation in clastic content of these two stratigraphic horizons is definitive when coupled with the HCL test. Consequently, the staff considers RAI 2.5.1-9 to be resolved.

2.5.1.4.2.3.2 Stratigraphic Data from Boreholes

In RAIs 2.5.1-35 and 2.5.1-48, the staff asked the applicant to explain how Rupert (1988) derived his lithologic descriptions for the deep petroleum exploration wells that penetrated the Avon Park Formation in the site vicinity, and to present the criteria used to conclude that washout of soft carbonate layers produced the no-return and no-recovery zones noted by Rupert (1988) in the logs for these deep petroleum wells, rather than open or filled dissolution voids. In the responses to RAIs 2.5.1-35 and 2.5.1-48, the applicant stated that Rupert (1988) relied on Vernon (1951) and the Florida Geological Survey (FGS) for lithologic descriptions, and noted that none of the driller's logs from the FGS reported dissolution voids in the upper 305 m (1,000 ft) of the deep petroleum exploration boreholes, which passed through the Avon Park Formation. In addition to the previous deep petroleum test wells, which penetrated the Avon Park Formation, the applicant analyzed cores from the LNP site taken from borings that penetrated to 152 m (500 ft) below the ground surface as part of the LNP site geotechnical investigations program and noted that Eocene (55.8 to 33.9 Ma) formations in the site area, including the Avon Park Formation, commonly contain interbedded hard (dolomite) and soft (weathered limestone) horizons. The applicant acknowledged that such a stratigraphic sequence requires careful drilling methods to avoid low core recovery, and reported that initial drilling in the Avon Park Formation often resulted in variable recovery rates. To determine that the poor recovery zones resulted from washout of soft carbonate horizons, the applicant drilled six supplemental boreholes at the LNP site located to be offset approximately 1.5 m (5 ft) from the position of the initial site characterization boreholes. The applicant used controlled coring techniques to improve core recovery and documented the presence of soft zones, rather than dissolution voids, at depth. Based on these field data, the applicant concluded that the no-return and no-recovery zones detected in core samples from the LNP site resulted from washout of soft horizons in the normal stratigraphic sequence.

Based on review of the applicant's responses to RAIs 2.5.1-35 and 2.5.1-48, as well as direct examination of lithologic and geophysical logs for the deep petroleum wells the applicant provided plus review of core samples from grout test holes during the September 2009 site audit and of boring logs, core photographs, and written core descriptions from the six supplemental "offset" boreholes located at the LNP site during February 2010, the staff concludes that the missing zones in the Avon Park Formation are due to washouts of softer horizons in the normal stratigraphic sequence, rather than to large open or filled dissolution voids. Examination of cores from the grout test holes and of data from the six "offset" supplemental boreholes did not reveal the presence of large dissolution voids in the Avon Park Formation at the LNP site. The offset boreholes used minimal down-pressures, lower drilling fluid pressures, slower drilling rates, and a larger diameter core barrel specifically to improve core recovery and determine if missing zones in the Avon Park Formation resulted from voids or washout of soft zones in the normal stratigraphic sequence. These supplemental data documented that the no-recovery zones logged in the initial site characterization boreholes resulted from washout of soft zones, rather than dissolution voids. Therefore, the staff draws this conclusion because the preponderance of field data from boreholes at LNP Units 1 and 2, including the data directly reviewed by NRC staff, strongly supports this interpretation. Consequently, the staff considers RAIs 2.5.1-35 and 2.5.1-48 to be resolved.

Based on review of LNP COL FSAR Section 2.5.1.2.3, review of the applicant's responses to RAIs 2.5.1-8, 2.5.1-9, 2.5.1-35 and 2.5.1-48 and the changes implemented in FSAR Section 2.5.1.2, and independent review of borehole data as described above, the staff finds that the applicant provided a complete and accurate description of site vicinity and site area stratigraphy in support of the LNP COL application.

2.5.1.4.2.4 Site Vicinity and Site Area Structural Geology

In FSAR Section 2.5.1.2.4, the applicant discussed structural geology of the LNP site vicinity and site area. The applicant stated that recent geologic mapping shows no faults within the 40-km (25-mi) radius site vicinity, and that no known structural features have been identified at the site location within a 1-km (0.6-mi) radius of the site. The applicant also discussed regional fracture systems in Florida as initially defined by Vernon (1951), the relationship between those regional fracture systems and smaller-scale fracture patterns near the site, and differing interpretations of the Ocala Platform. The staff focused the review of FSAR Section 2.5.1.2.4 on the characteristics of regional and local fracture systems, including the relationships between fractures and surficial features related to karst development; origin of the Ocala Platform; and postulated tectonic structures in the LNP site vicinity.

2.5.1.4.2.4.1 Observed Fracture Patterns

In RAI 2.5.1-2, the staff asked the applicant to explain whether the local (i.e., outcrop-scale) fractures observed and measured in the site area, referred to as a "subset" of the regional fracture system by the applicant, are smaller-scale fractures that parallel regional fracture trends. The staff also asked the applicant to discuss whether these local fractures exercise control on dissolution. In response to RAI 2.5.1-2, the applicant indicated that "local fractures" refer to vertical outcrop-scale fractures, such as those observed in the Avon Park Formation both along the Waccasassa River and at the abandoned Gulf Hammock quarry, while "regional fractures" are those linear features, identified by Vernon (1951) using aerial imagery, which extend across the site region. Due to the similarity in orientations of these two different scales of fractures, the applicant concluded that the local fractures can be interpreted as smaller-scale features, which reflect the regional fracture system identified by Vernon (1951). Based on field observations of local fracture systems and examination of regional lineament patterns on aerial imagery, the applicant also concluded that local and regional fracture systems strongly influence local dissolution because fractures act as conduits for groundwater flow, and that fractures exercise strong control on dissolution in the site vicinity and site area, particularly where the vertical fractures intersect near-horizontal bedding planes. The applicant cited Dr. T. Scott (personal communications, June 2009) of the FGS, who stated, based on his field observations, that fractures are common in limestone and dolostone quarries in the site vicinity; that fractures in limestones are noticeably enlarged by dissolution; and that fractures in dolostones show less enlargement and limited void development due to dissolution. The applicant also noted consistency between orientations of aligned wetlands and surface depressions associated with mapped lineaments at the LNP site and trends of fracture sets observed and measured in the CR3 site excavations.

Based on review of the applicant's response to RAI 2.5.1-2, as well as direct observation and measurement of local fracture systems along the Waccasassa River in April 2009 and at the Gulf Hammock quarry in September 2009, which enabled a comparison of orientations of the regional and local fracture systems, the staff concludes that outcrop-scale fractures in the Avon Park Formation share a common orientation and likely represent two different scales of the same fracture system. Based on strong confirmation from field data, the staff concludes that fractures exercise strong control on dissolution, and consequently karst development, in the site vicinity and site area, particularly where vertical fractures intersect horizontal bedding planes. Consequently, the staff considers RAI 2.5.1-2 to be resolved.

In RAI 2.5.1-10, the staff asked the applicant to explain the basis for distinguishing "primary" and "secondary" fractures at both local and regional scales, and to provide further description of the local fracture sets in regard to their characteristics and possible origin (i.e., tectonic or non-tectonic). In response to RAI 2.5.1-10, the applicant stated that "primary" and "secondary," as applied to local fractures observed in the Avon Park Formation at the Gulf Hammock quarry and along the Waccasassa River, reflect fracture prominence and frequency to be consistent with descriptions of "major" (primary) and "minor" (secondary) regional fracture sets inferred from photolineament analysis. That is, primary, or major, fractures are most prominent and occur most frequently at both local and regional scales. Based on field measurements of fractures in outcrops at the Gulf Hammock quarry and along the Waccasassa River, the applicant reported that the dominant strike directions of the primary fracture sets are N39W and N51E (i.e., orthogonal fractures), while the secondary fracture sets trend approximately N-S and E-W. The applicant noted that it is not currently possible to define a specific mechanism for development of the primary and secondary fractures sets.

Based on the applicant's response to RAI 2.5.1-10, as well as direct observation and measurement of the local fracture systems in outcrops of the Avon Park Formation along the Waccasassa River in April 2009 and at the Gulf Hammock quarry in September 2009, which provided independent observation of the field relationships, the staff concludes that the distinction the applicant made between primary and secondary fractures is correct and that the orientations of these fracture sets are N39W and N51E (primary) and N-S and E-W (secondary). Based on direct observation of field characteristics of the fractures, the staff also concludes that a specific causative mechanism for the fracture sets cannot be deduced from the field relationships and has not currently been determined by area experts. Consequently, the staff considers RAI 2.5.1-10 to be resolved.

In RAI 2.5.1-39, the staff asked the applicant to discuss the relationship of fractures mapped at the CR3 site to fracture patterns expected to occur at the LNP site, and to regional fracture systems that control stream drainage and sinkhole alignment patterns; to compare the spacing of regional fracture sets with spacing of fractures measured at the CR3 site and anticipated to occur at the LNP site; and to explain why fracture sets interpreted as conjugate, implying that they are tectonically-induced shear fractures, geometrically appear to be orthogonal. In response to RAI 2.5.1-39, the applicant acknowledged that characterization of fractures is important for identifying and mitigating potential hazards related to karst. The applicant reported that the FSAR for CR3 did not provide detailed information about spacing or orientations of fractures observed in the excavation for that plant, so that comparisons of fracture data from the CR3 site could not be made with regional fracture sets or fractures expected at the LNP site. However, the applicant noted that orientations of lineaments defined by slope breaks or alignment of circular depressions and associated wetlands in the LNP site vicinity are consistent with trends of the fracture sets reported for the CR3 site excavation. The applicant stated that, although bedrock exposures at the LNP site location are insufficient to evaluate length or spacing of fracture sets in the Avon Park Formation at the site, fracture spacing observed at the Gulf Hammock quarry and along the Waccasassa River are likely representative of fracture spacing at the LNP site. The applicant also indicated that the fracture sets initially referred to as conjugate are orthogonal based on observed fracture geometry. The applicant incorporated changes in LNP COL FSAR Section 2.5.1.2.4 to further clarify fracture characteristics.

Based on review of the applicant's responses to RAIs 2.5.1-2, 2.5.1-10, and 2.5.1-39 and the changes implemented in LNP COL FSAR Section 2.5.1.2.4, as well as independent review of existing fracture data and direct field observation and measurement of fracture patterns in outcrops of the Avon Park Formation along the Waccasassa River (April 2009) and at the Gulf Hammock quarry (September 2009), the staff concludes that orientations and spacings of fractures observed in the site vicinity and site area and suggested by lineament studies likely reflect orientations and spacing of fractures at the LNP site. The staff also concludes that fractures exercise strong control on dissolution and karst development. The staff draws these conclusions because the preponderance of data from both outcrop studies and lineament analyses do not indicate unique fracture orientations and spacings for the site vicinity, site area, or site location, and do support the interpretation that fractures control dissolution and karst development. Consequently, the staff considers RAIs 2.5.1-2, 2.5.1-10, and 2.5.1-39 to be resolved.

2.5.1.4.2.4.2 The Ocala Arch (or Platform)

In RAI 2.5.1-11, the staff asked the applicant to discuss the origin of the Ocala arch (or platform) in regard to whether it is tectonic or non-tectonic, including any possible association of regional and local fracture sets with development of the Ocala arch. In response to RAI 2.5.1-11 regarding origin of the Ocala arch, the applicant stated that the consensus of knowledgeable FGS geologists is that this feature developed due to differential subsidence, erosion, and sedimentation, rather than as a result of tectonic uplift, and the applicant provided information to document this interpretation. The applicant stated further that the Ocala arch does not exhibit fracture patterns that are uniquely different from the prominent regional fracture systems, which occur statewide, or from the local fracture patterns, which reflect the same trends as the regional fracture systems and are, therefore, related to its genesis.

Based on review of the applicant's response to RAI 2.5.1-11, including independent review of information from area experts at the FGS provided by the applicant, the staff concludes that the applicant provided current information regarding origin of the Ocala arch and possible association of fractures with the platform. The staff draws this conclusion because the applicant assessed the existing data related to origin of the Ocala arch with due consideration for the most current interpretations by FGS geologists, the recognized area experts, who interpret the Ocala arch as non-tectonic in origin and state that regional and local fracture patterns are not unique to the platform. Consequently, the staff considers RAI 2.5.1-11 to be resolved.

2.5.1.4.2.4.3 Postulated Faults and Identification Criteria

In RAI 2.5.1-38, the staff asked the applicant to summarize the information leading to the conclusion that no faults occur within the site vicinity, and to discuss the criteria applied to distinguish faults from fractures. In response to RAI 2.5.1-38, the applicant summarized pertinent data collected by FGS geologists, including geologic maps, cross sections, and structure contour maps, used to determine that no faults occur in the site vicinity (e.g., a statewide 1:750,000-scale geologic map and cross sections from Scott and others, 2001; a 1:126,720-scale geologic map of Levy County from Campbell, 1992; a 1:500,000-scale geologic map of the Floridian aquifer system from Knapp, 1979; and structure contour maps developed by Arthur and others, 2008). None of these data sources developed by area experts from the FGS showed discontinuities or anomalies resulting in the interpretation of surface or subsurface faults in the site vicinity. However, the applicant noted that Arthur et al. (2008) postulated two short segments of a northwest-trending subsurface fault just outside the site vicinity, located about 42 km (26 mi) southeast of the LNP site at its nearest point, based on abrupt changes in thickness in the Suwannee Limestone, as suggested by their structure contour maps. The applicant indicated that there is no surface expression of this postulated fault documented in the current literature cited by the applicant and, if it exists, it is pre-Quaternary (> 2.6 Ma) in age since there is no disruption of Quaternary sediments overlying the inferred fault. Finally, the applicant defined several standard criteria used to distinguish faults from fractures in the site vicinity and site area, all of which depend on finding geologic evidence of displacement along the fault surface as indicated by the presence of sheared materials; visible fault offset or offset inferred from geologic map data; anomalies that suggest truncation or offset of geologic materials; or deposits and geomorphic surfaces disrupted by folding or tilting. By applying these criteria and considering the data collected by FGS geologists, the applicant concluded that no faults occur within the site vicinity.

Based on review of the applicant's response to RAI 2.5.1-38, as well as independent review of pertinent published literature provided by the applicant and data related to structural geology of the site vicinity and site area, including borehole information, the staff concludes that no current data support the existence of faults in the LNP site vicinity or site area. The staff makes this conclusion because the information provided by the applicant, and reviewed by the staff, documented the geologic map data used to assess the presence of faulting. In addition, the staff concludes that the criteria the applicant used to assess the presence of faulting in the site area and site vicinity are the standard criteria for recognition of faults based on field data. Consequently, the staff considers RAI 2.5.1-38 to be resolved.

Based on review of LNP COL FSAR Section 2.5.1.2.4, the applicant's responses to RAIs 2.5.1-2, 2.5.1-10, 2.5.1-11, 2.5.1-38, and 2.5.1-39 and associated changes implemented in FSAR Section 2.5.1.2.4, as well as independent review of pertinent literature cited by the applicant and data and direct field observation of fractures in the Avon Park Formation, the staff finds that the applicant provided a complete and accurate description of structural geology of the site vicinity and site area in support of the LNP COL application.

2.5.1.4.2.5 Site Location Geology

In FSAR Section 2.5.1.2.5, the applicant discussed geology of the site location, including geomorphology, stratigraphy, and karst development. The staff focused the review of FSAR Section 2.5.1.2.5 on the applicant's discussion of factors governing karst development and possible size of subsurface dissolution cavities at the site location.

2.5.1.4.2.5.1 Potential for Rapid Groundwater Flow Conduits

In RAI 2.5.1-31, the staff asked the applicant to discuss available information related to the existence of underground conduits capable of accommodating rapid groundwater flow at or near the LNP site. In RAI 2.5.1-47, the staff asked the applicant to provide a reference for a statement included in the response to RAI 2.5.1-31 that no springs of any noticeable magnitude exist within the LNP site vicinity. In responses to RAIs 2.5.1-31 and 2.5.1-47, the applicant stated that the LNP site lies in a zone of very low recharge, and cited Upchurch (personal communication, 2009) to document the absence of significant springs within the outcrop area of the Avon Park Formation, including the site vicinity. The applicant presented a map modified from Maddox (1993), which shows that no known caves occur within the outcrop area of the Avon Park Formation in Levy and Citrus Counties. Scott and others (2004) reported only two small springs near the LNP site, namely Big King and Little King Springs, which lie to the north-northwest and within 8 km (5 mi) of the site. The applicant concluded that few voids, and no large ones, occurred in the LNP site characterization borings, and reiterated that the upper 150 m (50 ft) of the Avon Park Formation consists primarily of dolomitized limestone (i.e., dolostone), which is less susceptible to dissolution than pure limestone.

Based on review of the applicant's responses to RAIs 2.5.1-31 and 2.5.1-47, as well as independent examination of cores and borehole logs from the LNP site in September 2009 and February 2010 that did not reveal interconnected underground voids or extensive fractures in the subsurface, the staff concludes that no evidence exists for interconnected underground conduits capable of accommodating rapid groundwater flow at or near the LNP site. The staff draws this conclusion because no springs of significant magnitude occur at or near the LNP site, and the site characterization core samples directly examined by staff did not contain interconnected or large voids in the subsurface. Consequently, the staff considers RAIs 2.5.1-31 and 2.5.1-47 to be resolved.

2.5.1.4.2.5.2 Size of Subsurface Dissolution Cavities

The staff requested that the applicant clarify information related to the possible maximum size of subsurface dissolution cavities as provided in a supplemental discussion of the potential for karst development at the site location (Progress Energy, 2008). In RAIs 2.5.1-5 and 2.5.1-7, the staff asked the applicant to address the uncertainty in the estimate of a maximum lateral extent for dissolution cavities of 3 m (10 ft), as cited in the supplemental discussion, and to discuss the potential for coalescing dissolution cavities at depth below LNP Unit 1 or LNP Unit 2. In responses to RAIs 2.5.1-5 and 2.5.1-7, the applicant stated that conservative parameters applied in the analysis of size of subsurface karst features based on grout uptake volume accounted for uncertainties in the subsurface data used to estimate the maximum size of dissolution voids. These conservative parameters included increasing grout volumes used in the void size analysis above the grout uptake volumes calculated from borehole data, specifically by 50-percent for vertical fractures and 100-percent for horizontal bedding planes. The use of the parameters resulted in the applicant defining a dissolution cavity with a maximum lateral dimension of 3 m (10 ft), whereas the maximum void size calculated from actual borehole data was 1.6 m (5.3 ft) in lateral extent. The applicant pointed out that the size of the dissolution cavity used in the analysis is 1.9 times the size of the cavity calculated from borehole data, and thus concluded that the estimate of maximum size of subsurface dissolution cavities presented in the supplemental discussion was conservative. The applicant noted that the degree of dolomitization of the Avon Park Formation, a process, which lowers the likelihood of dissolution, decreased the potential for coalescence of subsurface dissolution cavities. The applicant provided information documenting the fact that dolomites dissolve less readily than pure limestones in response to RAI 2.5.1-1 discussed below in SER Section 2.5.1.4.2.6, "Site Area Geologic Hazard Evaluation."

Based on the review of the applicant's responses to RAIs 2.5.1-5 and 2.5.1-7, as well as independent examination of supporting field data from grout test cores in September 2009 and the six "offset" boreholes drilled using controlled boring techniques to improve core recovery and enable assessment of subsurface dissolution cavities and fractures in February 2010, the staff concludes that the estimate of a maximum void size of 3 m (10 ft) in lateral extent is conservative. The staff makes this conclusion because the preponderance of field data indicates that large subsurface dissolution cavities do not occur in the Avon Park Formation at the site location. The supporting field data examined during the September 2009 site audit specifically showed grout uptake only in a single vertical fracture intersected by one of the test grouting boreholes, and no large dissolution cavities occurred in any of the boreholes. The supporting data examined in February 2010 enabled the staff to conclude these data indicate that the low recovery horizons noted in the initial site characterization boreholes for LNP Units 1 and 2 (as examined by staff during the site visit in April 2009) mark soft zones in the normal stratigraphic sequence, rather than large subsurface dissolution cavities. Consequently, the staff considers RAIs 2.5.1-5 and 2.5.1-7 to be resolved.

In RAIs 2.5.1-12 and 2.5.1-46, the staff asked the applicant to discuss what the scale of surficial features may suggest in regard to a maximum lateral dimension for dissolution voids in the subsurface. In responses to RAIs 2.5.1-12 and 2.5.1-46, the applicant indicated that surface morphology of the LNP site is characterized by shallow depressions, classified as solution sinkholes, which vary in size from small, well-defined depressions less than 50 m (64 ft) in diameter and 1 to 2 m (2 to 6 ft) in depth to large, irregular, shallow depressions ranging up to 600 m (2,000 ft) wide. Based on Sinclair and Stewart (1985), the applicant reported that the diameter of these shallow, surficial solution sinkholes observed at the LNP site is not indicative of the size of expected subsurface karst features. Following Sinclair and Stewart (1985), the applicant stated that dissolution is most active at the limestone surface where dissolution features develop, commonly along fractures that allow water to easily percolate into the subsurface, dissolve the limestone, and transport insoluble residues, such that these features indicate shallow dissolution only. The applicant further indicated that deep dissolution does not commonly occur because subsidence of the soil layer occurs as the surface of the limestone dissolves and seals the bottom of the shallow depression, forming a marsh or lake in the depression. The applicant stated that this shallow dissolution process produced the undulating topography characterized by the shallow depressions, which are common over large parts of Florida and which dominate the LNP site.

Based on review of the applicant’s responses to RAIs 2.5.1-12 and 2.5.1-46, as well as independent review of Sinclair and Stewart (1985) and other pertinent published literature cited by the applicant, the staff concludes that the shallow solution sinkhole depressions, which dominate the surface of the LNP site, are surficial sinkholes that do not reflect deep dissolution cavities. The staff makes this conclusion because experts in the region have documented this interpretation based on borehole data that do not reveal deep dissolution cavities beneath these solution sinkholes. Consequently, the staff considers RAIs 2.5.1-12 and 2.5.1-46 to be resolved.

Based on review of LNP COL FSAR Section 2.5.1.2.5, the applicant’s responses to RAIs 2.5.1-5, 2.5.1-7, 2.5.1-12, 2.5.1-31, 2.5.1-46, and 2.5.1-47, as well as independent review of pertinent literature cited by the applicant and data and direct observation of grout test cores in September 2009 and examination of information from the six “offset” boreholes drilled using controlled boring techniques to improve core recovery in February 2010, the staff finds that the applicant provided a complete and accurate description of site location geology in support of the LNP COL application.

2.5.1.4.2.6 Site Area Geologic Hazard Evaluation

FSAR Section 2.5.1.2.6 presents an evaluation of the geologic hazards at the LNP site. The applicant noted that the LNP site is located in an area of infrequent and low seismicity, and that no capable tectonic sources occur in the site area. The applicant did not indicate whether field reconnaissance studies or literature searches cited by the applicant were performed to determine if paleoliquefaction features (i.e., indicators of prehistoric earthquake activity) occur in the site region, vicinity, or area. The applicant concluded that the only geologic hazard identified in the LNP site area is potential surface deformation resulting from carbonate dissolution and collapse or subsidence related to karst development.

The staff focused the review of FSAR Section 2.5.1.2.6 on qualification of the dissolution rates cited for development of karst at the LNP site, and whether paleoliquefaction features may exist in the site region, site vicinity, or site area as indicators of prehistoric seismic events.

2.5.1.4.2.6.1 Proposed Dissolution Rates

In RAI 2.5.1-1, the staff asked the applicant to summarize the technical basis for the dissolution rates cited in the LNP COL FSAR, and to document the statement in the FSAR that dolomitized limestone dissolves more slowly than pure limestone. In response to RAI 2.5.1-1, the applicant indicated that a comparison of the more dolomitized Avon Park Formation with the less dolomitized Ocala Formation at the CR3 site provided the dissolution rate of less than 1E-4 percent per year proposed for the Avon Park Formation at the LNP site. The applicant stated that the dissolution rate for the Ocala Formation at the CR3 site, 1E-4 percent per year, calculated out to 6E-3 percent over the projected 60-year life of that plant. Regarding the degree of dolomitization of the Avon Park Formation at the LNP site, which converts limestone to dolomite, the applicant reported that 18 of 20 samples from the LNP site analyzed during LNP site characterization investigations exhibited a high degree of dolomitization, containing less than 50 percent calcium carbonate ($CaCO_3$). The applicant reported that Easterbrook (1999) documented that about 60 percent $CaCO_3$ is necessary to form karst, and about 90 percent may be required to fully develop karst. Also citing Easterbrook (1999), the applicant stated that dolomites, composed of calcium-magnesium carbonate [$CaMg(CO_3)_2$], have a lower permeability than non-dolomitized limestones. This characteristic diminishes dissolution and karst formation. The applicant concluded that the potential for dissolution and karst formation at the LNP site during the life of the plant is not significant, and added that a monitoring program would be established for the LNP plant to confirm this low dissolution rate as part of the groundwater monitoring program.

Based on review of the applicant's response to RAI 2.5.1-1, as well as an independent review of the references cited therein, the staff concludes that there is a strong technical basis for the proposed low dissolution rate at the site location. The staff draws this conclusion because characterization of the Avon Park Formation indicates that this unit is dolomitized at depth, and there is a preponderance of published information to document that dolomites and dolomitic limestones have much lower dissolution rates than pure limestones. Consequently, the staff considers RAI 2.5.1-1 to be resolved. The staff further concludes that the only geologic hazard identified in the LNP site area is potential non-tectonic surface deformation resulting from collapse or subsidence related to karst development. The staff addresses this potential hazard in SER Section 2.5.3.4.8.

2.5.1.4.2.6.2 Paleoliquefaction Features

In RAI 2.5.1-41, the staff asked the applicant to discuss the efforts undertaken to document the presence or absence of paleoliquefaction features in the site region, site vicinity, and site area, or to explain why such efforts were not thought to be necessary. In response to RAI 2.5.1-41, the applicant stated that no published or unpublished reports reviewed during site characterization or preparation of FSAR Section 2.5 identified paleoliquefaction features in the LNP site region. In addition, based on discussions with Dr. T. Scott of the FGS (personal communications, 2009), the applicant confirmed that no paleoliquefaction features have been reported anywhere in Florida. The applicant also discussed observations made during field reconnaissance in the LNP site vicinity and site area, which resulted in the suggestion that detailed studies, would not likely provide data useful for evaluating the occurrence, location, or size of prehistoric earthquakes in the LNP site vicinity and area. The applicant indicated that a paucity of exposures and limited stratigraphy favorable for liquefaction in the site vicinity, including along major drainages, rendered it difficult to document the presence or absence of paleoliquefaction features. Therefore, based on existing information documenting that no reported paleoliquefaction features occur in the site region and that Florida currently has a low risk of earthquakes, communications with a knowledgeable expert from the FGS indicating that no paleoliquefaction features have been observed in Florida, and the existence of only sparse exposures, which lack materials favorable for liquefaction, the applicant stated that detailed paleoliquefaction studies were not performed to assess the possibility of prehistoric earthquakes in the site region, site vicinity, or site area.

Based on review of the applicant's response to RAI 2.5.1-41, the staff concludes that paleoliquefaction features are not likely to exist in the site region, site vicinity, or site area. The staff draws this conclusion because investigations by experts knowledgeable about the geology and seismicity of Florida have not demonstrated the existence of paleoliquefaction features anywhere in the State of Florida. In addition, the Florida platform on which the LNP site is located reflects regional tectonic quiescence since the Cretaceous (145.5 Ma) as discussed in FSAR Section 2.5.1.1.2, and there is no geologic or geomorphic evidence of Quaternary (2.6 Ma to present) faulting as discussed in FSAR Section 2.5.3. Consequently, the staff considers RAI 2.5.1-41 to be resolved.

Based on review of FSAR Section 2.5.1.2.6 and the applicant's responses to RAIs 2.5.1-1 and 2.5.1-41, the staff finds that the applicant provided a complete and accurate description of potential geologic hazards in the site area in support of the LNP COL application.

2.5.1.4.2.7 Site Engineering Geology Evaluation

FSAR Section 2.5.1.2.7 discusses site engineering geology, including engineering behavior of soil and rock; zones of alteration, weathering, and structural weakness; karst features; and deformation zones. The applicant indicated that FSAR Section 2.5.4 discusses engineering behavior of soil and rock materials at the site, and that, if any karst features occur in the LNP foundation rocks, then they will be addressed through appropriate design considerations as explained in that FSAR section. The applicant stated that no zones of structural weaknesses (e.g., extensive fracture zones or faults) have been identified at the LNP site; that the Avon Park Formation does exhibit weathering alteration and varying degrees of dissolution; and that, with the exception of possible paleosinkholes, no deformation zones have been encountered.

Based on the review of FSAR Section 2.5.1.2.7, as well as independent review of current literature cited by the applicant related to geologic and geotechnical characteristics of the LNP site, the staff finds that the applicant provided a complete and accurate description of site engineering geology in support of the LNP COL application.

2.5.1.5 *Post Combined License Activities*

There are no post-COL activities related to FSAR Section 2.5.1. However, in SER Section 2.5.3.4.8 ("Potential for Surface Deformation at the Site"), the staff identified a geologic mapping

License Condition related to FSAR Section 2.5.3.8.1 as the responsibility of the COL licensee. SER Section 2.5.3.5 addresses this License Condition.

2.5.1.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The staff confirmed that the applicant addressed the required information related to basic geologic and seismic characteristics, and that there is no outstanding information expected to be addressed in the LNP COL FSAR related to these characteristics. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the staff has reviewed the information in LNP COL 2.5-1 and finds that the applicant provided a thorough characterization of basic geologic and seismic information for the LNP site, as required by 10 CFR 100.23 and 10 CFR 52.79 (a)(1)(iii). In addition, the staff concludes that the applicant identified and appropriately characterized all seismic sources significant for determining the GMRS, or SSE, for the COL site, in accordance with NRC regulations provided in 10 CFR 100.23 and 10 CFR 52.79(a)(1)(iii) and the guidance provided in RG 1.208. Based on the applicant's geologic investigations of the site region and site area, the staff concludes that the applicant properly characterized regional and site lithology, stratigraphy, geologic and tectonic history, and structural geology, as well as subsurface soil and rock units at the site. The staff also concludes that there is no potential for the effects of human activity (i.e., mining activity or ground water injection or withdrawal) to compromise the safety of the site. Therefore, the staff concludes that the proposed COL site is acceptable from the standpoint of basic geologic and seismic information and meets the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23.

2.5.2 Vibratory Ground Motion

2.5.2.1 *Introduction*

The vibratory ground motion is evaluated based on seismological, geological, geophysical, and geotechnical investigations carried out to determine the site-specific ground motion response spectrum (GMRS), which must meet the regulations for the Safe Shutdown Earthquake (SSE) provided in 10 CFR 100.23. The GMRS is defined as the free-field horizontal and vertical GMRS at the plant site. The development of the GMRS is based upon a detailed evaluation of earthquake potential, taking into account the regional and local geology, Quaternary tectonics, seismicity, and site-specific geotechnical engineering characteristics of the site subsurface material. The specific investigations necessary to determine the GMRS include the seismicity of the site region and the correlation of earthquake activity with seismic sources. Seismic sources are identified and characterized, including the rates of occurrence of earthquakes associated with each seismic source. Seismic sources that have any part within 320 km (200 miles) of the site must be identified. More distant sources that have a potential for earthquakes large enough to affect the site must also be identified. Seismic sources can be capable tectonic sources or seismogenic sources. The review covers the following specific areas: (1) seismicity, (2) geologic and tectonic characteristics of the site and region, (3) correlation of earthquake activity with seismic sources, (4) probabilistic seismic hazard analysis and controlling earthquakes, (5) seismic wave transmission characteristics of the site, (6) site-specific ground motion response spectrum, and (7) any additional information requirements prescribed within the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52, "Licenses, certifications, and approvals for nuclear power plants."

2.5.2.2 *Summary of Application*

Section 2.5 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.5.2 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.5.2, the applicant provided site-specific information to address the following:

AP1000 COL Information Item

- LNP COL 2.5-2

The applicant provided additional information in LNP COL 2.5-2 to address COL Information Item 2.5-2. LNP COL 2.5-2 addresses the provision for site-specific information related to vibratory ground motion aspects of the site including: seismicity, geologic and tectonic characteristics, correlation of earthquake activity with seismic sources, PSHA, seismic wave transmission characteristics and the SSE ground motion.

- LNP COL 2.5-3

The applicant provided additional information in LNP COL 2.5-3 to resolve COL Information Item 2.5-3, which addresses the provision for performing site-specific evaluations, if the site-specific GMRS at foundation level exceed the response spectra in AP1000 DCD Figures 3.7.1-1 and 3.7.1-2 at any frequency, or if soil conditions are outside the range evaluated for the AP1000 DCD.

2.5.2.2.1 Seismicity

FSAR Section 2.5.2.1 describes the development of a current earthquake catalog for the LNP Units 1 and 2 site. The applicant used the methodology provided in RG 1.208 by starting with the EPRI- Seismicity Owners Group (SOG) historical earthquake catalog (EPRI NP-4726-A, 1988), which is complete from 1627 to 1984. The applicant updated EPRI-SOG's historical earthquake catalog with seismicity from 1985 through December 2006 using current seismicity catalogs. The current seismicity catalogs include data from the Advanced National Seismic System (ANSS), the International Seismological Centre (ISC), Virginia Tech Seismological Observatory's Southeastern U.S. Seismic Network, and the U.S. Geological Survey (USGS) National Earthquake Information Center (NEIC). The applicant deleted duplicate entries for the final updated catalog and converted the different magnitude scales used by the catalogs to body wave magnitude (m_b), which is the scale used in the EPRI-SOG catalog.

The applicant's seismicity catalog update includes the seismicity data from the Bellefonte Geotechnical, Geological, and Seismological (GG&S) earthquake catalog (TVA, 2006) extended to latitude 23°N and longitude 107°W, and through December 2006. This extended coverage includes the LNP Units 1 and 2 320-km (200-mi) site radius and seismicity throughout the Gulf of Mexico. These were not included in the Bellefonte GG&S earthquake catalog. The geographic distribution of earthquakes in the applicant's updated earthquake catalog is provided in SER Figure 2.5.2-1.

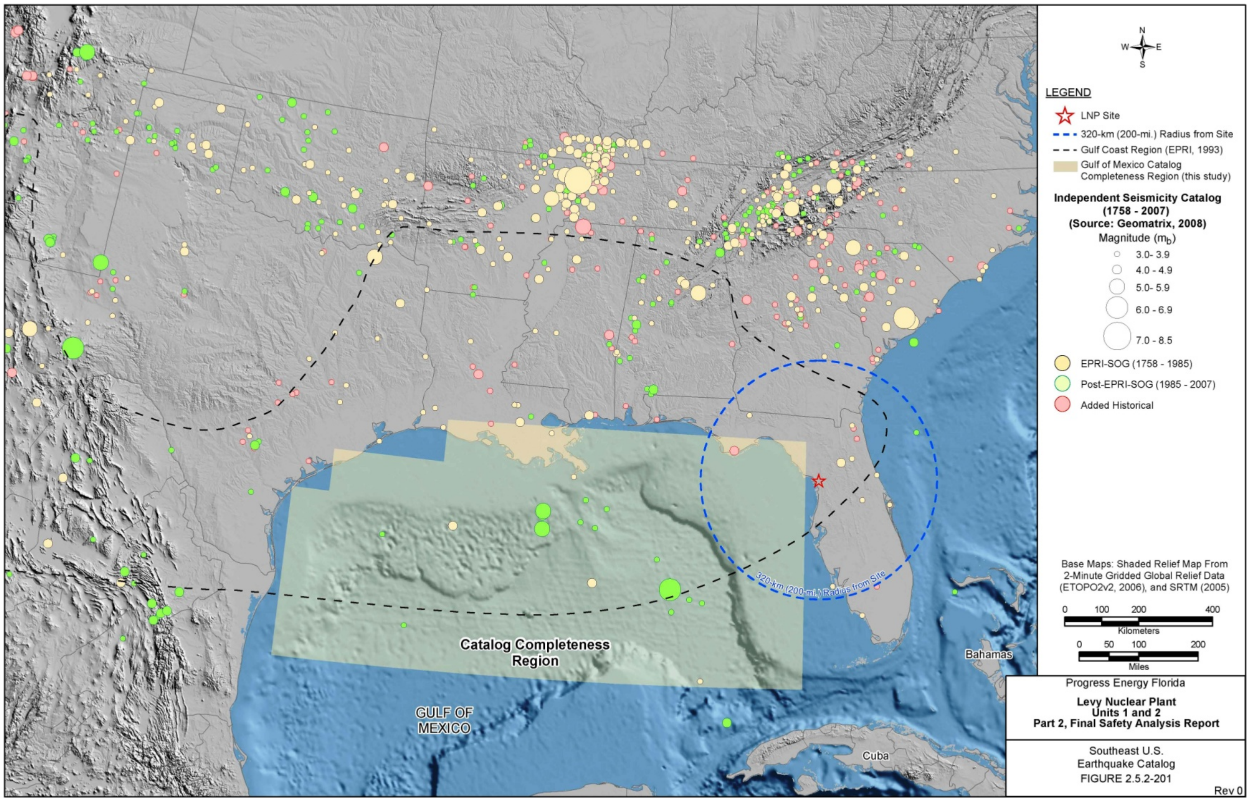


Figure 2.5.2-1. Topography and Bathymetry Map Showing the Applicant's Updated Earthquake Catalog and the Location of the LNP Site (FSAR Figure 2.5.2-201)

2.5.2.2.1.1 Earthquakes that May Influence Seismic Hazard at the LNP Site

In FSAR Section 2.5.2.1.2, the applicant identified three regions where earthquakes occur that may significantly influence the seismic hazard at the LNP Units 1 and 2 site. The first is the LNP Units 1 and 2 site region, which encompasses seismicity within the 320-km (200-mi) site radius. The second area includes the earthquakes in the Gulf of Mexico and the third region contains the historic earthquakes of the Charleston, South Carolina region. The applicant's description of these regions is summarized below.

2.5.2.2.1.1.1 Earthquakes within the 320-km (200-mi) LNP Site Region

The applicant noted that there are fifteen earthquakes with m_b greater than or equal to 3 located within 320 km (200 mi) of the LNP Units 1 and 2 site, which is shown in SER Figure 2.5.2-1. As described in the applicant's updated earthquake catalog, event magnitudes do not exceed m_b of 4.3 and the earthquakes occurred between the years 1826 and 2006. Out of these earthquakes thirteen earthquakes have magnitudes (m_b) between 3 and 4 ($3 \leq m_b < 4$) and two earthquakes have magnitudes greater than 4 ($4 \leq m_b < 4.3$). The applicant noted that estimates of Modified Mercalli Intensity (MMI) and strong motion records are not available for these earthquakes. The largest earthquake within the LNP site region occurred on January 13, 1879, near St. Augustine, Florida at a distance of 76 km (47 mi) from the LNP site, and at a magnitude of m_b 4.3.

Earthquakes in the Gulf of Mexico

As shown in SER Figure 2.5.2-1, the southwestern portion of the 320-km (200-mi) site region extends into the Gulf of Mexico. However, the original EPRI-SOG earthquake catalog covers only a small portion of the Gulf of Mexico along the US coastline. The applicant updated the original EPRI-SOG catalog with seismicity within the Gulf of Mexico between the latitude 24° North (N) to 32° N and longitude 100° West (W) to 83° W. This update was prompted by the occurrence of two moderate-sized seismic events in the Gulf region. These two events (m_b 4.9 on February 10, 2006 and m_b 6.0 on September 10, 2006) are shown in SER Figure 2.5.2-1. The applicant calculated the magnitude of these events from the average of catalog reported m_b estimates and m_b estimates converted from other magnitude scales as reported in FSAR Table 2.5.2-201. The applicant noted that the m_b 6.0 earthquake is the closest Gulf of Mexico earthquake to the LNP Units 1 and 2 site. The effect of this earthquake was felt in Crystal River, Florida, which is located approximately 16 km (10 mi) south of the LNP site. Reports from Crystal River indicate an MMI of IV, which generally means that the ground motion resulting from the earthquake was moderately felt by Crystal River residents but no damage was sustained. To characterize periods of catalogue completeness for the Gulf of Mexico, the applicant adopted the procedure used in the EPRI-SOG study and divided the seismicity catalog into time frames and the event magnitude scale into intervals and determined a probability of completeness for each interval. The applicant's probabilities of detection for the Gulf of Mexico Completeness Region are listed in FSAR Table 2.5.2-211.

Historic Earthquakes of the Charleston, South Carolina Region

The September 1, 1886, Charleston, South Carolina earthquake is the largest (m_b 6.8) known event to occur in the southeastern United States. According to the LNP updated earthquake catalog, the event was located 494 km (307 mi) north of the LNP Units 1 and 2 site. Ground motion associated with the event was felt throughout northern Florida and the effects of several aftershocks were felt as far as Jacksonville, Florida, which is located approximately 217 km (135 mi) northeast of the LNP Units 1 and 2 site.

2.5.2.2.2 Geologic and Tectonic Characteristics of the Site and Region

FSAR Section 2.5.2.2 describes the original EPRI-SOG (EPRI, 1988) seismic source models that contribute to 99 percent of the total hazard at the LNP Units 1 and 2 site. These contributing EPRI-SOG sources are from the 1989 EPRI-SOG PSHA study. In that study, EPRI-SOG analyzed seismic source models for the Crystal River Unit 3 Nuclear Generating Plant (CR3) located about 15 km (10 mi) from the LNP Units 1 and 2 site. The applicant began its assessment of seismic sources at the LNP Units 1 and 2 site using the sources found to contribute to 99 percent of the total hazard at the CR3 site based on the 1989 EPRI-SOG study. EPRI-SOG designated six earth science teams (ESTs) to develop seismic source models for the Central and Eastern United States (CEUS), which were completed in 1986. The applicant also reviewed available geological, seismological, and geophysical data since the late 1980's to evaluate the need for modifications to the original EPRI-SOG ESTs' seismic source models. SER Section 2.5.2.2.4 describes the applicant's sensitivity studies of these potential source zone updates as well as potential new seismic sources.

2.5.2.2.2.1 Summary of EPRI Seismic Sources

Consistent with RG 1.208, the applicant used the 1986 EPRI-SOG seismic source model for the CEUS as a starting point for its seismic source characterization of the LNP Units 1 and 2 site. The 1986 EPRI-SOG seismic source model is comprised of input from six independent ESTs that include the Bechtel Group, Dames & Moore, Law Engineering, Rondout Associates, Weston Geophysical Corporation, and Woodward-Clyde Consultants (WCC). The 1989 EPRI-SOG study (EPRI, 1989) subsequently incorporated each of the EST models into a PSHA for nuclear power plant sites in the CEUS. FSAR Section 2.5.2.2.1 and FSAR Tables 2.5.2-202 through 2.5.2-207 detail the primary seismic sources developed by each of the six ESTs that contributed to 99 percent of the total hazard at the CR3 site and were assessed by the applicant for contributing to the hazard at the LNP Units 1 and 2 site. The seismic source models developed by the six ESTs are briefly described below.

2.5.2.2.2.1.1 Bechtel Group

Five Bechtel Group seismic source zones contributed to 99 percent of the total hazard at the CR3 site. The Charleston Area (H) and Faults (N3) sources represent locations for the 1886 Charleston earthquake and have an assigned maximum m_b of 7.4. The Atlantic Coastal Region (BZ4) has an assigned maximum m_b of 7.4 and is a background source that encompasses eastern Mesozoic basins and the Charleston area sources. The Gulf Coast Zone (BZ1) is a background source that encompasses most of the site region and extends from western Texas to eastern Florida, while the Southern Appalachians Region (BZ5) covers the large area of the southern Appalachians to the north of the site. Both the Gulf Coast Zone and the Southern Appalachians Region have an assigned maximum m_b of 6.6.

2.5.2.2.2.1.2 Dames & Moore

Six seismic sources defined by Dames & Moore contributed to 99 percent of the total hazard at the CR3 site. The South Cratonic Margin (41) covers the continental margin region and has an assigned maximum m_b of 7.2. The Southern Appalachian Mobile Belt source (53) has an assigned maximum m_b of 7.2 and characterizes rocks that have undergone multiple periods of deformation. The Charleston region (54) and Charleston Mesozoic Rift (52) sources represent sources for the 1886 Charleston earthquake and the surrounding area and have assigned maximum m_b of 7.2. The Southern Coastal Margin source (20) is a large background source that extends from Mexico, along the Texas coastal plain to eastern Florida with an assigned maximum m_b of 7.2. The Paleozoic (Appalachian) Fold Belt source (4) covers the folded mountain belt from New York to Alabama and has an assigned maximum m_b of 7.2.

2.5.2.2.2.1.3 Law Engineering

Eight Law Engineering seismic source zones contributed to 99 percent of the total hazard at the CR3 site. The Eastern Basement (17) source encompasses a larger area of buried Precambrian-Cambrian normal faults, where the assigned maximum m_b is 6.8. The Eastern Basement Background source (217) covers a pattern of magnetic anomalies and a negative Bouguer gravity signature, where the assigned maximum m_b is 5.7. The Reactivated Normal Faults source (22) has a maximum m_b of 6.8 and describes seismicity along the Eastern Seaboard region. The Charleston source (35) represents a source for the 1886 Charleston earthquake and the surrounding area and has an assigned maximum m_b of 6.8. The Mesozoic Basin source (8) encompasses the northeast-trending troughs of Triassic to early Jurassic age with a maximum m_b of 6.8. The South Coastal Block Zone (126) encompasses most of the site region and extends from Mexico, through Texas, to eastern Florida and has an assigned maximum m_b of 4.9. The Eastern Piedmont source (107) is located north of the LNP site region and has a maximum m_b of 6.8. The Brunswick source (108) is a background source that characterizes a basement terrane and has an assigned maximum m_b of 6.8.

2.5.2.2.2.1.4 Rondout Associates

Six seismic source zones defined by Rondout Associates contributed to 99 percent of the total hazard at the CR3 site. The Southern New York–Alabama Lineament source (13) characterizes a change in the regional magnetic anomaly pattern in basement rocks with an EPRI assigned maximum m_b of 6.5. The Charleston source (24) represents a source for the 1886 Charleston earthquake, including the Ashley River and Woodstock Faults, using a maximum m_b of 7.0. The Southern Appalachians source (25) characterizes anomalies associated with the New York-Alabama lineament and has an assigned maximum m_b of 7.0. The South Carolina Zone source (26) has a maximum m_b of 6.8 and encompasses cross-cutting fracture zones on the aeromagnetic map of South Carolina. The Appalachian Crust source (49) encompasses the location of the LNP site and is a background source of crust made of an accretionary terrane formed after the Precambrian. This source has an assigned maximum m_b 5.8. The Gulf Coast to Bahamas Fracture Zone (51) encompasses most of the site region and extends from Mexico and Texas to eastern Florida. The maximum assigned m_b for this zone is 5.8.

2.5.2.2.2.1.5 Weston Geophysical Corporation

Six Weston Geophysical Corporation seismic source zones contributed 99 percent of the total hazard at the CR3 site. The New York–Alabama–Clingman Block source (24) characterizes a linear block of seismicity in the Southern Appalachians, where the assigned maximum m_b is 6.6. The Charleston source (25) is localized on and around the city of Charleston, South Carolina and represents a source for the 1886 Charleston earthquake using a maximum m_b of 7.2. The South Carolina Zone source (26) describes the larger region of the state of South Carolina using an assigned maximum m_b of 7.2. The Southern Appalachian source (103) is a background zone located north of the site region and has a maximum assigned m_b of 6.6. The Southern Coastal Plains source (104) is a background source of the south coastal plain seismicity zone and has an assigned maximum m_b of 7.2. The Gulf Coast Zone (107) is a large areal source that extends from Mexico and Texas to eastern Florida and encompasses most of the site region. The maximum m_b assigned is 6.0.

2.5.2.2.2.1.6 Woodward-Clyde Consultants

Four Woodward-Clyde Consultants seismic source zones were found to contribute 99 percent of the total hazard at the CR3 site. The Greater South Carolina sources (29, 29A, and 29B) characterize seismicity in South Carolina, Georgia and western North Carolina with an assigned maximum m_b of 7.4. The Charleston source (30) represents a local source for the 1886 Charleston earthquake with a maximum m_b of 7.5. The Blue Ridge Zone and Alternative sources (31 and 31A) extend from the south to the central Appalachians and have a maximum m_b of 7.0. The Crystal River source (B36) encompasses most of the state of Florida, including the CR3 site and LNP sites and has maximum m_b of 6.5.

2.5.2.2.2.2 Post-EPRI Seismic Source Characterization Studies

In accordance with the guidance in RG 1.208, the applicant reviewed seismic source characterization studies published since the original EPRI-SOG (1988) study to assess the need to update the 1986 EPRI-SOG seismic source parameters. Based on LNP's updated seismicity catalog and the results of several post-ERPI studies (Frankel et al., 2002; SCDOT, 2003; and SNC, 2006), the applicant updated the EPRI-SOG (1988) characterizations of the Charleston seismic source zone and the Gulf Coastal Source Zones (GCSZ).

2.5.2.2.2.2.1 USGS National Seismic Hazard Mapping Project

The applicant stated that as part of the 2002 update of the National Seismic Hazard Maps, the USGS developed a model of the Charleston source that incorporates available data regarding recurrence, M_{max}, and geometry of the source zone. The USGS model used two equally weighted source geometries: (1) an areal source enveloping most of the tectonic features and liquefaction data in the greater Charleston area, and (2) a north-northeast-trending elongated areal source enveloping the southern half of the southern segment of the proposed East Coast fault system. For maximum moment magnitudes (M_{max}), the study defines a distribution of M 6.8 (0.20), 7.1 (0.20), 7.3 (0.45), 7.5 (0.15). For recurrence, USGS (Frankel et al., 2002) adopted a mean paleoliquefaction-based recurrence interval of 550 years and represented the uncertainty with a continuous lognormal distribution. The applicant chose to update the EPRI Charleston seismic source using the Updated Charleston Seismic Source (UCSS) model presented in Southern Nuclear Company's (SNC) ESP (SNC, 2007) application for VEGP Units 3 and 4, which is discussed below.

2.5.2.2.2.2.2 Updated Charleston Seismic Source Zone

The site of the 1886 large-magnitude Charleston, SC earthquake lies approximately 494 km (307 mi) north of the LNP Units 1 and 2 sites. The applicant included the UCSS zone developed by SNC (2006), because the Charleston area is the closest principle source of seismic activity to the LNP Units 1 and 2 site. SNC's new zone accounts for updated information regarding the location, size, and rate of earthquake occurrence for large-magnitude earthquakes in the vicinity of Charleston, SC. The UCSS model includes four possible source regions as shown in FSAR Figure 2.5.2-213. In the model, the four seismic sources are treated as potential zones capable of producing large earthquakes. The size of the characteristic earthquake is assumed to vary from magnitude M_{max} 6.7 to 7.5 in each of these four alternative source zones. The applicant used these seismic source geometries and modeled the occurrence of large repeated earthquakes in the Charleston region. Since the distance between the updated Charleston sources and the LNP site is relatively far at 494 km (307 mi), the applicant updated only the EPRI-SOG (1988) source models for the large-magnitude earthquakes within the Charleston source zone. The applicant assumed that smaller magnitude earthquakes of less than 6.7 at such large distances would not significantly affect the seismic hazard at the LNP Units 1 and 2 sites. Therefore the applicant retained the 1986 EPRI-SOG Charleston sources but limited the Mmax in those sources to m_b 6.6.

2.5.2.2.2.2.3 Gulf Coastal Source Zone

The applicant's updated earthquake catalog includes the two 2006 Gulf of Mexico earthquakes that exceed the bounds of the upper end of the M_{max} distributions for a few EPRI-SOG source models for the Gulf Coast. These earthquakes are the February 10, 2006, m_b of 4.9 earthquake and the September 10, 2006, m_b of 6.0 earthquake. Because of this, the applicant revised five of the six ESTs' M_{max} distributions for GCSZ background sources that contain the LNP site. The applicant's updates to the GCSZs are the same as those made in the South Texas Project (STP) Units 3 and 4 COL application (STPNOC, 2008) and are listed in SER Table 2.5.2-1. The applicant concluded that the increases in M_{max} adequately accounts for the February 10 and September 10, 2006, earthquakes and any potential association between the earthquakes within the Gulf of Mexico and proposed normal faults along the edge of the continental shelf.

Table 2.5.2-1. EPRI-SOG EST GCSZ updates from the STP Unit 3 and 4 COLA. (FSAR Table 2.5.2-209)

EPRI-SOG EST	SOURCE	DESCRIPTION	PROBABILITY OF ACTIVITY	M_{max} DISTRIBUTIONS EPRI-SOG (1989) m_b [WEIGHTS]	UPDATED M_{max} DISTRIBUTIONS STP Unit 3 and 4 (STPNOC, 2008) m_b [WEIGHTS]
Bechtel Group	BZ1	Gulf Coast	1.0	5.4 [0.1] 5.7 [0.4] 6.0 [0.4] 6.6 [0.1]	6.1 [0.1] 6.4 [0.4] 6.6 [0.5]

Table 2.5.2-1. EPRI-SOG EST GCSZ updates from the STP Unit 3 and 4 COLA. (FSAR Table 2.5.2-209)

EPRI-SOG EST	SOURCE	DESCRIPTION	PROBABILITY OF ACTIVITY	M_{max} DISTRIBUTIONS EPRI-SOG (1989) m_b [WEIGHTS]	UPDATED M_{max} DISTRIBUTIONS STP Unit 3 and 4 (STPNOC, 2008) m_b [WEIGHTS]
Dames & Moore	20	South Coastal Margin	1.0	5.3 [0.8] 7.3 [0.2]	5.5 [0.8] 7.3 [0.2]
Law Engineering	126	South Coastal Block	1.0	4.6 [0.9] 4.9 [0.1]	5.5 [0.9] 5.7 [0.1]
Rondout Associates	51	Gulf Coast to Bahamas Fracture Zone	1.0	4.8 [0.2] 5.5 [0.6] 5.8 [0.2]	6.1 [0.3] 6.3 [0.55] 6.5 [0.15]
Weston Geophysical Corporation	107	Gulf Coast	1.0	5.4 [0.71] 6.0 [0.29]	6.6 [0.89] 7.2 [0.11]
Woodward-Clyde Consultants	B43	Central US Backgrounds	NA	4.9 [0.17] 5.4 [0.28] 5.8 [0.27] 6.5 [0.28]	No update

2.5.2.2.3 Correlation of Earthquake Activity with Seismic Sources

FSAR Section 2.5.2.3 describes the correlation of updated seismicity with the EPRI-SOG seismic source models. As described above, the applicant created an updated seismicity catalog covering the LNP site region as part of FSAR Section 2.5.2.1.1. The applicant compared the distribution of earthquake epicenters from the updated seismicity catalog with the seismic sources characterized by each of the EPRI-SOG ESTs, and drew the following conclusions:

- There are no new identifiable seismic sources or active geologic features within the 320-km (200-mi) radius site region and all earthquake activity follows the pattern identified in the EPRI-SOG characterizations. The updated earthquake catalog has spatial patterns and estimated seismicity occurrence rates similar to that of the EPRI-SOG earthquake catalog. Therefore, the applicant made no significant revisions to the EPRI-SOG seismic source geometries or recurrence rates.

- The two 2006 earthquakes that occurred in the Gulf of Mexico are not covered by the M_{max} used by some of the EPRI-SOG ESTs for their Gulf Coast seismic source models. As a result, the applicant revised some of the ESTs' M_{max} distributions for its Gulf Coast models.

- The 1886 Charleston, South Carolina earthquake is the largest historical earthquake to occur in the southeastern United States and the applicant considered this event the closest principle source of seismic activity to the LNP Units 1 and 2 site. The EPRI-SOG teams considered the 1886 earthquake, but more recent studies have further studied alternative source locations, M_{max} values, and large-magnitude recurrence rates. Therefore, the applicant incorporated the findings of the other studies to more adequately characterize the Charleston seismic zone.

2.5.2.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquake

FSAR Section 2.5.2.4 presents the results of the applicant's probabilistic seismic hazard analysis (PSHA) for the LNP Units 1 and 2 site. In performing its PSHA, the applicant followed the guidance provided in RG 1.208 to determine the seismic hazard curves and controlling earthquakes for the LNP Units 1 and 2 site. The applicant based its analyses on the original EPRI hazard study (1989) and used the seismic sources identified in EPRI-SOG's 1988 study and updated them as necessary. The PSHA curves generated by the applicant represented generic hard rock conditions characterized by a V_S in excess of 2.7 kilometers per second (km/s) (9,000 feet per second (fps)). The applicant also described the earthquake potential for the site in terms of the uniform hazard response spectra (UHRS) and the controlling earthquakes, the most likely earthquake magnitudes and source-site distances. The applicant determined the low- and high-frequency controlling earthquakes by deaggregating the PSHA curves at selected probability levels. Before determining the controlling earthquakes, the applicant updated five of the six GCSZ defined by EPRI (1989) and used the new ground motion models described below.

2.5.2.2.4.1 PSHA Inputs

Before performing the PSHA, the applicant updated the GCSZ inputs from the original 1989 EPRI study and used the updated EPRI (2004, 2006) ground motion models instead of the ground motion models used in the original EPRI study (1989).

2.5.2.2.4.1.1 Seismic Source Model

In order to conduct PSHAs and obtain the UHRS at the site, it is necessary to study the site location and its surrounding regions to determine geological and seismological properties, as outlined in RG 1.208. This requires identification of active seismic source zones in the area, compilation of a comprehensive list of earthquakes from the historical records and earthquakes that were recorded instrumentally, determination of earthquake occurrence rates in each of the seismic zones and their probability of occurrence, estimation of maximum magnitudes, and choosing ground motion prediction equations relevant to that region. As summarized above in SER Section 2.5.2.2.2, the seismic sources in the EPRI-SOG study consisted of six alternative seismic source models developed by six ESTs for the CR3 site. The applicant used these seismic source models as the starting point and updated them based on available new information. The applicant modified the EPRI-SOG source models as follows:

- For all sources identified as the Charleston source from each of the six EPRI EST models the Mmax was limited to m_b 6.6. The UCSS source model (SNC, 2006) was used to represent Charleston repeated large magnitude earthquakes (Mmax 6.7 to

7.5).Revised M_{max} distributions for five of the six EPRI EST seismic source models within the region of the GCSZ that contain the LNP Units 1 and 2 site, consistent with the updates made in the STP Units 3 and 4 COL application (STPNOC, 2008), as described in SER Table 2.5.2-1.

2.5.2.2.4.1.2 Ground Motion Models

The applicant used the ground motion models developed by the 2004 EPRI-sponsored study (EPRI, 2004) for the updated PSHA. The 2004 EPRI project reviewed the latest knowledge of CEUS ground motions. The study updated equations estimating median spectral acceleration and associated uncertainties as a function of earthquake magnitude and distance throughout the CEUS. The applicant modeled epistemic uncertainty using multiple ground motion equations with weights and multiple estimates of weighted aleatory uncertainty, which arises due to inherent randomness in data. The 2006 EPRI study found that the aleatory uncertainties were too large in EPRI (2004), thus resulting in an overestimation of seismic hazard. Therefore, the applicant used the 2004 EPRI ground motion models with the update of the 2006 EPRI aleatory uncertainty equations.

2.5.2.2.4.1.3 PSHA Sensitivity Analysis

Consistent with RG 1.208, the applicant evaluated potential impacts of new data and information in its seismic hazard calculations. The applicant provided sensitivity study results to evaluate the impacts of the proposed changes to the seismic parameters used in the PSHA calculations. These changes are categorized in four different areas: 1) selection of EPRI-SOG seismic sources near the LNP site; 2) updated source models for the Charleston, South Carolina region; 3) updated maximum magnitude distributions for the GCSZ; and 4) updated seismicity parameters for the GCSZ.

The applicant examined sources within the LNP 320-km (200-mi) site radius and sources at larger distances that could affect the site, such as Charleston, South Carolina. The sensitivity analysis assesses seismic hazard to establish any seismic source whose contribution to the total hazard exceeds 1 percent in the frequency of exceedance in the 10^{-4} and 10^{-5} range and, therefore, should be included in the hazard calculations.

The applicant concluded that the effect of both the Gulf of Mexico parameter updates and the Charleston source update resulted in an appreciable increase in the hazard. Therefore, the applicant incorporated these modifications into the updated PSHA for the LNP Units 1 and 2 site.

2.5.2.2.4.2 PSHA Methodology and Calculation

Using the updated EPRI-SOG seismic source characteristics and new ground motion models (EPRI, 2004) with updated uncertainties as inputs (EPRI, 2006), the applicant performed PSHA calculations for peak ground acceleration (PGA) and spectral acceleration at frequencies of 0.5, 1.0, 2.5, 5, 10, 25, and 100 Hertz (Hz). Following the guidance in RG 1.208, the applicant performed PSHA calculations assuming generic hard rock site conditions with a V_S of 2.8 km/s (9,200 fps).

2.5.2.2.4.3 PSHA Results

The applicant's PSHA results for the LNP Units 1 and 2 site are described in FSAR Section 2.5.2.4. The applicant performed the PSHA calculations using the EPRI-SOG seismic sources described in SER Section 2.5.2.2.2. Additionally, the applicant incorporated SNC's UCSS (2006) update of the large-magnitude Charleston, South Carolina source zone and the updates to the GCSZ, as described in SER Table 2.5.2-1. Site seismic hazard characteristics are quantified by the seismic hazard curves from the PSHA. The hazard curves were developed identifying and characterizing each seismic source that contributed to 99 percent of the seismic hazard at the LNP site. Using the hazard curves, the applicant developed UHRS, which are the spectral accelerations that have an equal likelihood of exceedance at different natural frequencies. FSAR Figures 2.5.2-226 through 2.5.2-232 illustrate the applicant's mean and 5^{th}, 16^{th}, 50^{th}, 84^{th}, and 95^{th} fractile hard rock hazard curves for the PGA and spectral acceleration at frequencies of 25, 10, 5, 2.5, 1, and 0.5 Hz. SER Figure 2.5.2-2 shows the mean UHRS for the 10^{-3}, 10^{-4}, 10^{-5}, and 10^{-6} annual frequencies of exceedance for hard rock conditions.

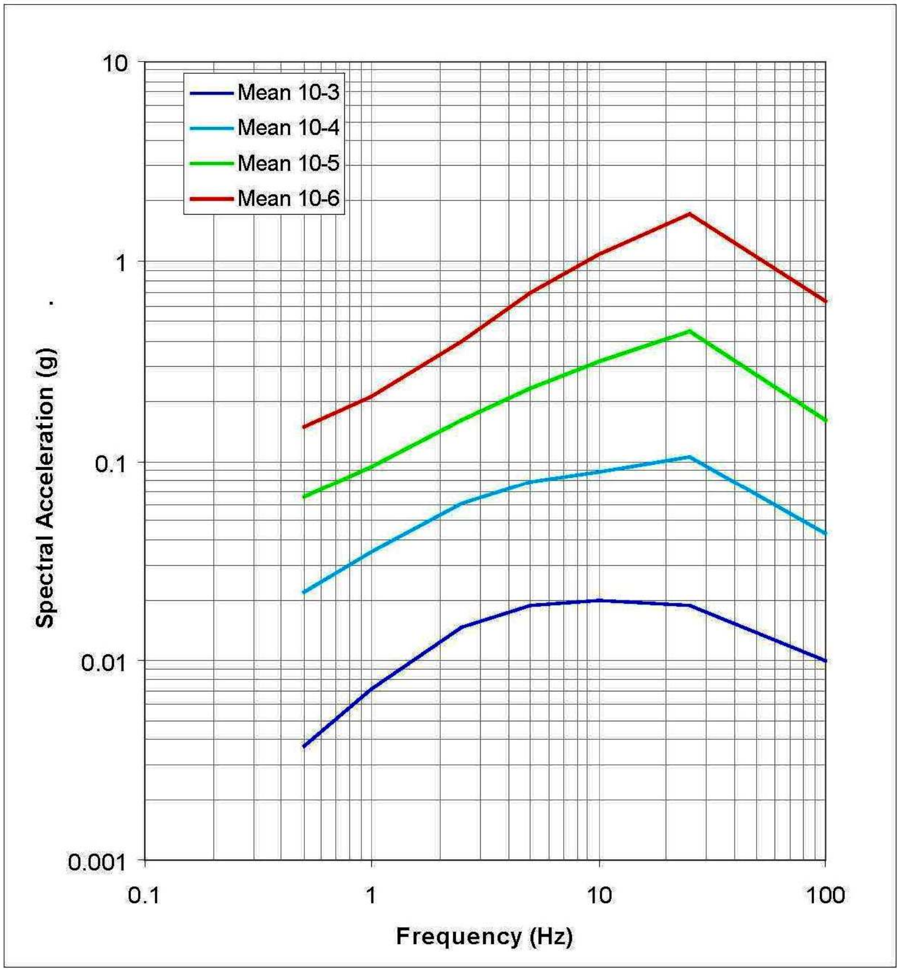


Figure 2.5.2-2. UHRS for the LNP 1 and 2 Site for Generic CEUS Hard Rock Conditions (FSAR Figure 2.5.2-238)

FSAR Section 2.5.2.4.4.2 describes the earthquake potential for the site in terms of the most likely earthquake magnitudes and source-to-site distances, which are referred to as ‘controlling earthquakes’. The applicant determined the controlling earthquakes that dominate low-frequencies (LF) and the high frequencies (HF), 1 and 2.5 Hz and 5 and 10 Hz, respectively. To determine the controlling earthquakes, the applicant deaggregated the PSHA at selected probability levels. The procedure the applicant used is outlined in RG 1.208. The applicant performed the deaggregation of the mean 10^{-3}, 10^{-4}, 10^{-5}, and 10^{-6} PSHA hazard results.

For use in the applicant’s site response analysis, which is summarized in the next SER section, the applicant developed “deaggregation earthquakes” (DE) from the controlling earthquakes. The DE parameters are listed in SER Table 2.5.2-2 and the applicant used these earthquakes to reflect the weighted distribution of earthquakes contributing to the hazard at the site. The applicant defined the weight of each DE by the relative contribution of the earthquake in a magnitude-distance domain of the total hazard. Using the EPRI median ground motions, the EPRI aleatory variability models, and the spectral shape functions from NUREG/CR-6728 CEUS ground motions, the applicant then developed smooth response spectra to represent each of the DE listed in SER Table 2.5.2-2.

Table 2.5.2-2. Deaggregation Earthquake Parameters (FSAR Table 2.5.2-221)

FREQUENCY RANGE (Hz)	MEAN ANNUAL FREQUENCY OF EXCEEDANCE	DEAGGREGATION EARTHQUAKES (DE)		
		MAGNITUDE (m_b)	DISTANCE (km [mi])	WEIGHT
1 and 2.5	10^{-4}	5.5 6.3 7.1	20.2 (12.5) 72 (45) 459 (285)	0.105 0.052 0.843
5 and 10	10^{-4}	5.4 6.2 7.1	27.7 (17.2) 70 (43) 455 (282)	0.320 0.077 0.603
1 and 2.5	10^{-5}	5.6 6.4 7.2	12.2 (7.5) 45 (28) 456 (283)	0.218 0.112 0.670
5 and 10	10^{-5}	5.4 6.3 7.2	13.6 (8.4) 29 (18) 453 (281)	0.615 0.156 0.229
1 and 2.5	10^{-6}	5.7 6.5 7.2	8.9 (5.5) 32 (20) 455 (282)	0.400 0.240 0.360
5 and 10	10^{-6}	5.4 6.4 7.2	8.9 (5.5) 15 (9.3) 450 (279)	0.681 0.297 0.022

2.5.2.2.3 Seismic Wave Transmission Characteristics of the Site

FSAR Section 2.5.2.5 describes the procedure the applicant used to assess the effects of soils on seismic wave transmission beneath the site. The hazard curves generated by the PSHA are defined for generic hard rock conditions characterized by a V_S of 2.8 km/s (9,200 fps). For the LNP Units 1 and 2 site, these hard rock conditions exist at a depth of 1,300 m (4,300 ft) beneath the ground surface, while materials with lower velocities exist in the upper 1,300 m (4,300 ft). To determine the near-surface UHRS, the applicant used Approach 2B outlined in NUREG/CR-6728). Following Approach 2B, the applicant: (1) developed soil models for the LNP Units 1 and 2 site; (2) randomized the soil profiles to account for variability; and (3) performed the final site response analysis.

In FSAR Section 2.5.2.5, the applicant described how it performed two sets of site response analyses. The applicant used one analysis to develop the site specific GMRS and the second analysis to perform the soil structure interaction (SSI) analyses. For the SSI analyses inputs, the applicant developed the performance based surface response spectra (PBSRS) and foundation input response spectra (FIRS). While the applicant described development of PBSRS and FIRS in FSAR Section 2.5.2.5, the summary and evaluation of the PBSRS, FIRS, and SSI analyses are described in SER Section 3.7.1.

2.5.2.2.3.1 Site Response Model

The applicant developed site-specific shallow V_S models for the upper 152 m (500 ft) based on the results of 18 compression (P) and shear (S) wave P-S suspension logging and downhole velocity survey wells and used four deep wells to make stratigraphic and velocity determinations to 1,676 m (5,500 ft) depth. The applicant estimated that the subsurface geology at the LNP Units 1 and 2 site consists of approximately 1,300 m (4,300 ft) of Cretaceous and Cenozoic limestone and dolomite and 1.8 m (6 ft) of Quaternary sands at the surface. The median shear wave velocity profile was added to the base of the shallow profiles to create the applicant's initial velocity profiles for site response analysis.

The applicant also estimated the parameter kappa (κ) as input into the site response analysis. Kappa is the near-surface damping parameter, which is an estimate of the dissipation of seismic energy of the site during an earthquake due to damping within soil layers and waveform scattering at layer boundaries. The applicant used two sets of modulus reduction and damping relationships to account for the potential of nonlinear behavior in the approximately 18.3 m (60 ft) of partly-to-moderately weathered limestone that occurs at a depth range of 48.8 to 67.1 m (160 to 220 ft). The remaining rock layers are assumed to behave linearly during seismic shaking.

The applicant's analysis resulted in the V_S profiles for the LNP Units 1 and 2 site illustrated in SER Figure 2.5.2-3, which were used in the applicant's GMRS analysis.

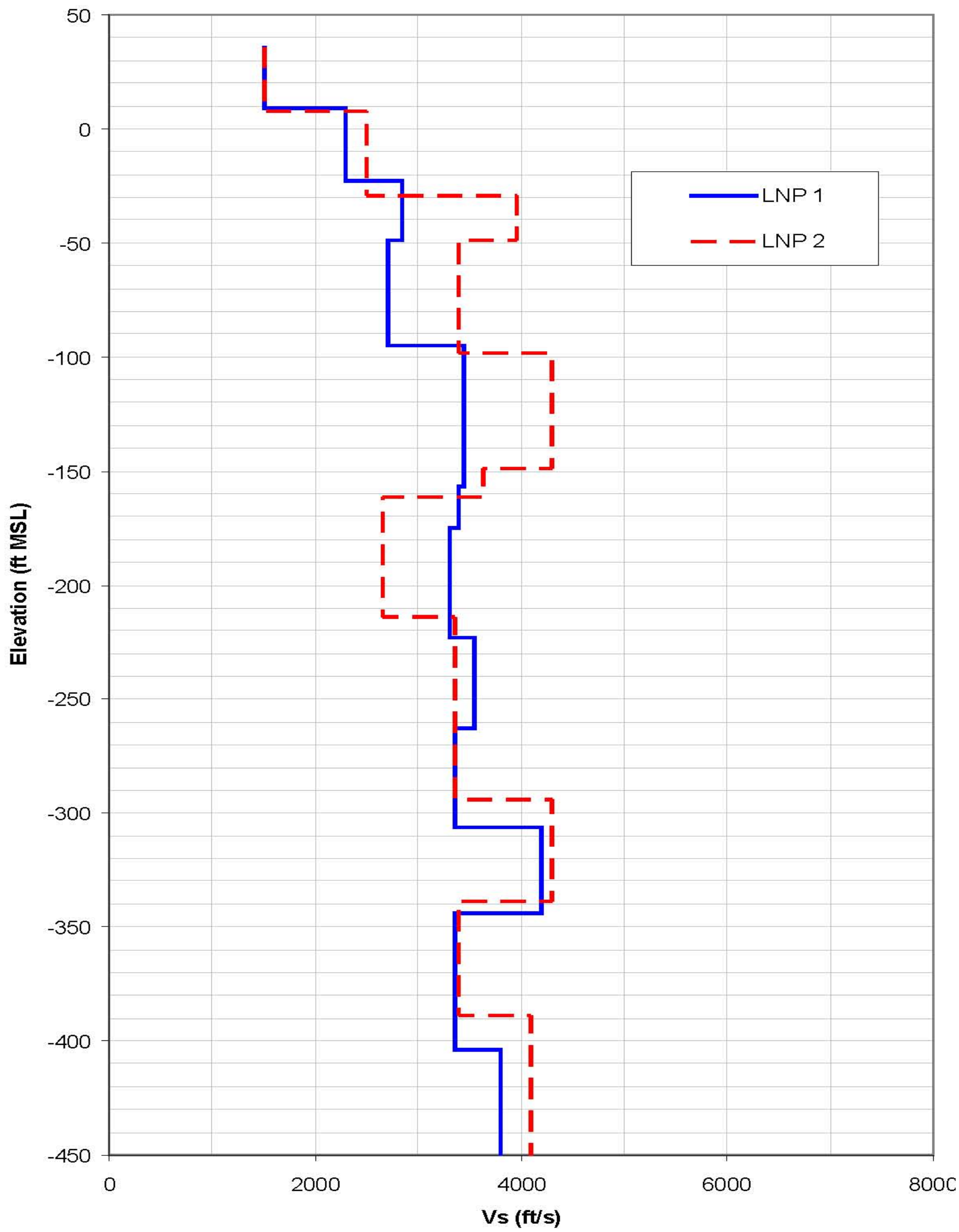


Figure 2.5.2-3. Shear Wave Velocity Profile for the LNP 1 and 2 Site Used in the GMRS Analysis
(FSAR Figure 2.5.2-254)

2.5.2.2.3.2 Site Response Methodology and Results

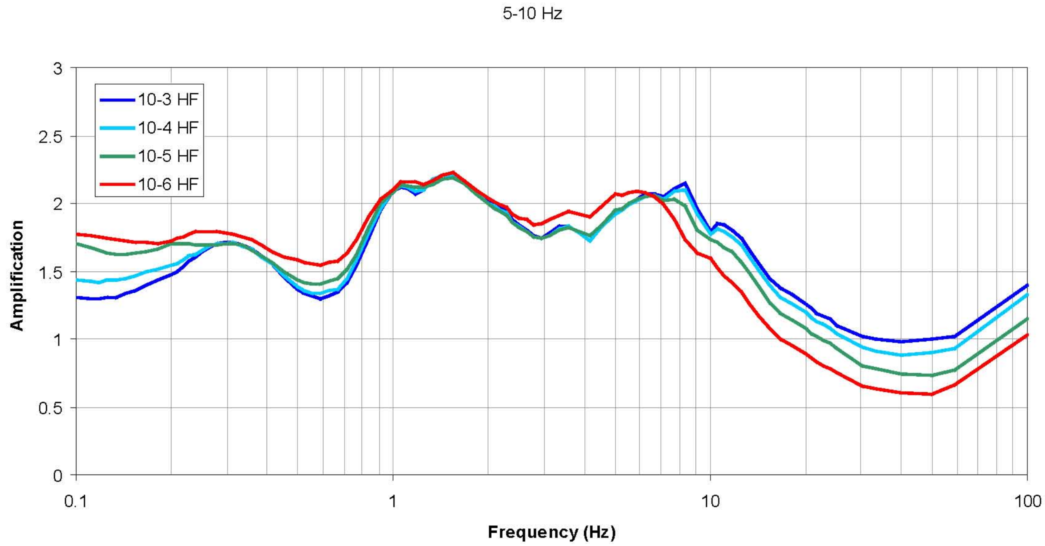
The applicant followed RG 1.208 and defined the site-specific GMRS at the top of the first competent layer. Since FSAR Section 2.5.4.5 states that the upper Quaternary sands have low velocity and are to be removed during construction, the reference point for the GMRS is taken to be the top of the calcareous silt unit S2, weathered limestone at an average elevation of 11 m (36 ft) using the North American Vertical Datum 1988 (NAVD88).

The applicant stated that once it determined the appropriate soil and rock dynamic properties, it modeled the variability present in the site data by randomizing the soil and rock V_S profiles, shear modulus reduction and damping values. The applicant generated 60 randomized profiles using the V_S correlation model developed by Silva et al. (1996). These artificial profiles represent the soil column from the top of bedrock to the ground surface.

The applicant developed response spectra for each controlling and deaggregation earthquake for two frequency ranges, HF (5 to 10 Hz) and LF (1 to 2.5 Hz), as defined in RG 1.208. The applicant developed 30 time histories from the sets given in NUREG/CR-6728 for each deaggregation earthquake spectrum. The applicant then scaled the selected time histories to match the target earthquake spectrum.

The applicant used the V_S profiles for the LNP Units 1 and 2 site as shown on SER Figure 2.5.2-3 to compute the site amplification functions for each of the spectrally matched time histories. For each hazard level (10^{-3}, 10^{-4}, 10^{-5}, and 10^{-6}) and for each controlling and deaggregation earthquake (HF and LF), the applicant paired the 60 randomized soil velocity profiles and the 60 randomized soil modulus reduction and damping curves with the 30 spectrally matched time histories. To compute the final site amplification effects, the applicant divided each output response spectrum (defined at the base of the nuclear island) by the corresponding hard rock input response spectrum and calculated the arithmetic mean of the 60 response spectral ratios.

The applicant compared the mean site amplification functions for the two GMRS profiles for the four levels of input motion (10^{-3}, 10^{-4}, 10^{-5}, and 10^{-6}). Because the comparison illustrated similarity in site amplification, the applicant used a single envelope amplification function for the LNP Units 1 and 2 sites. Those enveloped amplification functions for the four levels are plotted on SER Figure 2.5.2-4. The applicant again then enveloped and smoothed the amplification functions for the four ground motion levels, which is shown in FSAR Figure 2.5.2-280.



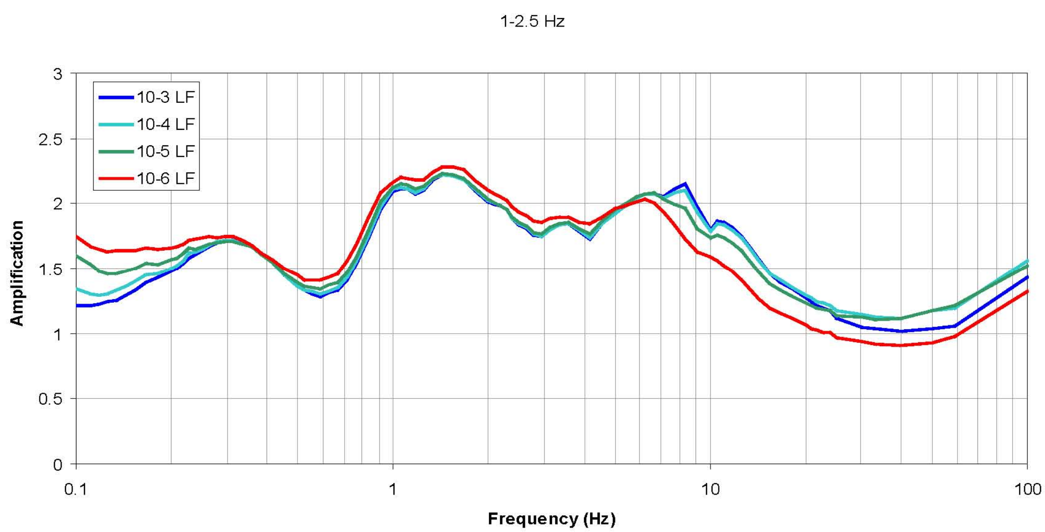


Figure 2.5.2-4. Envelope GMRS Amplification Functions for the LNP Unit 1 and 2 Site (FSAR Figure 2.5.2-278)

The applicant repeated the GMRS profile amplification function process described above for developing amplification functions at the base of the excavation creating the foundation input response spectra (FIRS). The analyses were performed including all material to design grade surface at elevation 15.5 m (51 ft.) NAVD88 and then extracting ground motion at -7.3 m (-24 m).Consistent with the GMRS analysis, a single envelope amplification function was developed for different hazard levels. The resulting FIRS amplification functions are plotted on SER Figure 2.5.2-5.

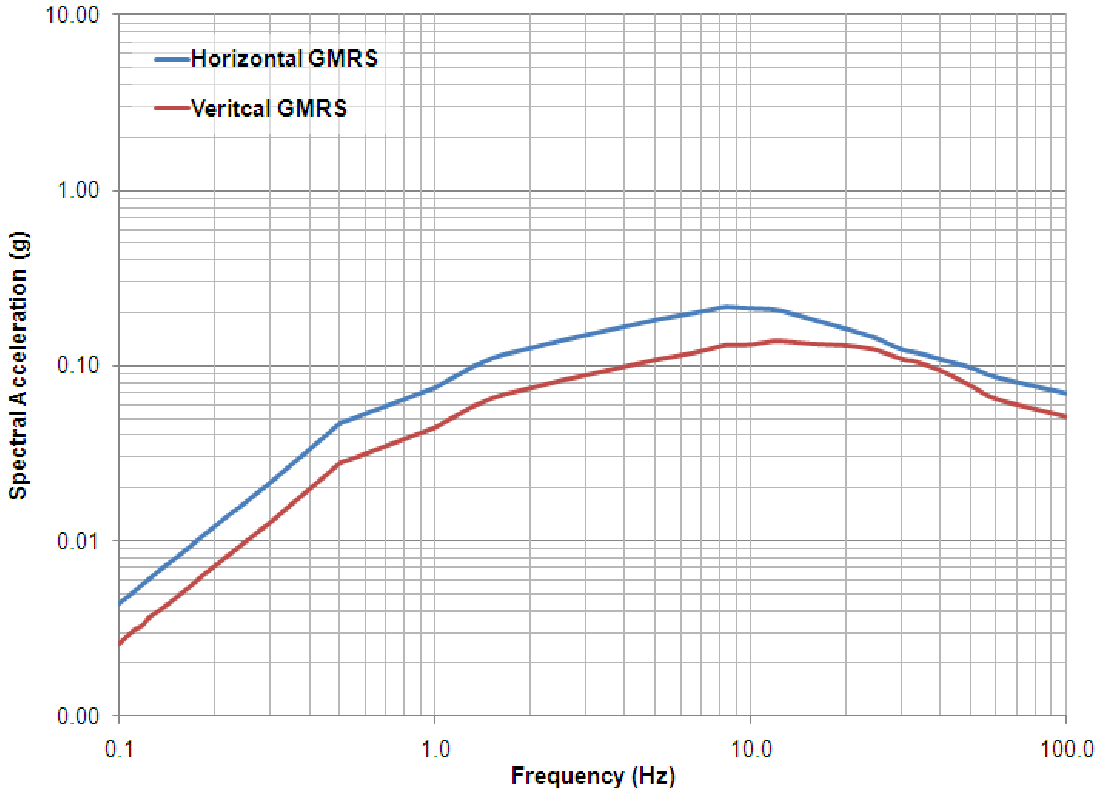


Figure 2.5.2-5. Horizontal and Vertical GMRS for the LNP Unit 1 and 2 Site (Reproduced from data in FSAR Table 2.5.2-226)

2.5.2.2.4 Ground Motion Response Spectra

FSAR Section 2.5.2.6 describes the method the applicant used to develop the horizontal and vertical site-specific GMRS. To obtain the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and in American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard 43-05, "American Society of Civil Engineers, Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities." The applicant developed the GMRS by scaling the rock controlling and deaggregation earthquakes and UHRS by the site amplification functions. The site-specific GMRS is defined at the top of the first competent layer at the elevation of 11 m (36 ft). The applicant developed the vertical GMRS by applying vertical-to-horizontal (V/H) response spectral ratios, based on NUREG/CR-6728, to the horizontal GMRS.

The applicant implemented the EPRI cumulative absolute velocity (CAV) model (EPRI, 2006) in a second set of PSHA calculations for the LNP Units 1 and 2 site. The method is described in RG 1.208 and is based on the probability that earthquakes of a given magnitude can produce damaging ground motions, where the damaging ground motion is defined as CAV exceeding 0.16 g-second. The EPRI CAV model results indicate that earthquakes of moment magnitude (M) less than 5 have little probability of producing ground motions greater than 0.16 g-second. The 10^{-4} surface UHRS with CAV is zero.

2.5.2.2.4.1 Horizontal GMRS

In FSAR Section 2.5.2.6.3, the applicant developed a horizontal, site-specific, performance-based GMRS using the method described in RG 1.208 and ASCE/SEI Standard 43-05. The performance-based method achieves the annual target performance goal (PF) of 10^{-5} per year for frequency of onset of significant inelastic deformation. This damage state represents a minimum structural damage state, or essentially elastic behavior, and falls well short of the damage state that would interfere with functionality. The horizontal GMRS for each spectral frequency, which meets the PF, is obtained by scaling the near-surface 10^{-4} UHRS by the design factor (DF):

$$DF = \max\left(1.0,\ 0.6(A_R)^{0.8}\right) \qquad \text{Equation (2.5.2-1)}$$

In SER Equation 2.5.2-1, the amplitude ratio, A_R, is given by the ratio of the 10^{-5} UHRS and the 10^{-4} UHRS spectral accelerations for each spectral frequency. When A_R exceeds 4.2, RG 1.208 specifies that the value of the GMRS is to be no less than 45 percent of the 10^{-5} UHRS. Since the 10^{-4} UHRS with CAV is 0, this criterion is used to define the horizontal GMRS. Finally, the applicant applied a scale factor to the horizontal GMRS. As described by the applicant in FSAR Section 2.5.2.5 and the staff in SER Section 3.7.1, the applicant developed site-specific scaled FIRS. The applicant calculated a scale factor of 1.212 such that the horizontal FIRS at 100 Hz is equal to 0.1 g as required by 10 CFR Part 50, Appendix S. To be consistent with the scaled FIRS, the applicant also applied the 1.212 scale factor to the horizontal GMRS. The resulting scaled spectrum is the applicant's horizontal GMRS, shown as the blue line in SER Figure 2.5.2-5 and these values are listed in FSAR Table 2.5.2-226.

2.5.2.2.4.2 Vertical GMRS

In FSAR Section 2.5.2.6.4, the applicant obtained the vertical GMRS by deriving V/H ratios and applying them to the applicant's final horizontal GMRS. The applicant calculated rock V/H ratios using spectral ratios from NUREG/CR-6728. NUREG/CR-6728 presents categories of V/H ratios for PGA less than 0.2 g, between 0.2 g and 0.5 g, and greater than 0.5 g. The applicant used ratios for PGA < 0.2 g, for the LNP Units 1 and 2 site. Since the applicant's best estimate of kappa for the LNP Units 1 and 2 site is intermediate between the Western United States (WUS) and CEUS, the applicant developed an intermediate V/H ratio for the LNP Units 1 and 2 site. FSAR Figure 2.5.2-295 shows the V/H spectral ratios for the WUS, CEUS, and the applicant's LNP intermediate values. The LNP vertical GMRS was then computed by multiplying the horizontal GMRS by the intermediate V/H ratio. The resulting vertical GMRS is shown as the red line in SER Figure 2.5.2-5 and values are listed in FSAR Table 2.5.2-226.

2.5.2.2.5 Sensitivity Study of CEUS Seismic Source Characterization Model

In January 2012, the NRC published NUREG-2115, "Central and Eastern United States Seismic Source Characterization for Nuclear Facilities." In FSAR Section 2.5.2.7, the applicant decribes its sensitivity study using the new seismic hazard model presented in NUREG-2115 and a modified CAV filter, as described in SECY-2012-0025 Enclosure 7, Attachment 1 to Seismic Enclosure 1. The staff's summary and evaluation of FSAR Section 2.5.2.7 is located in SER Section 20.1. Based on its sensitivity study, the applicant concluded that the scaled site-specific ground motions developed using the updated EPRI-SOG model with the CAV filter presented in FSAR Section 2.5.2.6 are appropriate for use as the design basis for the LNP site.

2.5.2.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed within the FSER related to the DCD.

In addition, the applicable regulatory requirements for reviewing the applicant's discussion of vibratory ground motion are as follows:

- 10 CFR 100.23 with respect to obtaining geologic and seismic information necessary to determine site suitability and ascertain that any new information derived from site-specific investigations does not impact the GMRS derived by a probabilistic seismic hazard analysis. In complying with this regulation, the applicant also meets guidance in RG 1.132 and RG 1.208.

- 10 CFR 52.79(a)(1)(iii), as it relates to consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity and period of time in which the historical data have been accumulated.

In addition, the related acceptance criteria from Section 2.5.2 of NUREG-0800 are summarized as follows:

- Seismicity: To meet the requirements in 10 CFR 100.23, this section is accepted when the complete historical record of earthquakes in the region is listed and when all available parameters are given for each earthquake in the historical record.

- Geologic and Tectonic Characteristics of Site and Region: Seismic sources identified and characterized by the Lawrence Livermore National Laboratory (LLNL) and the Electric Power Research Institute (EPRI) were used for studies in the CEUS in the past.

- Correlation of Earthquake Activity with Seismic Sources: To meet the requirements in 10 CFR 100.23, acceptance of this section is based on the development of the relationship between the history of earthquake activity and seismic sources of a region.

- Probabilistic Seismic Hazard Analysis and Controlling Earthquakes: For CEUS sites relying on LLNL or EPRI methods and data bases, the staff will review the applicant's PSHA, including the underlying assumptions and how the results of the site investigations are used to update the existing sources in the PSHA, how they are used to develop additional sources, or how they are used to develop a new data base.

- Seismic Wave Transmission Characteristics of the Site: In the PSHA procedure described in RG 1.208, the controlling earthquakes are determined for generic rock conditions.

- Ground Motion Response Spectra: In this section, the staff reviews the applicant's procedure to determine the GMRS.

In addition, the geologic and seismic characteristics should be consistent with appropriate sections from: RG 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants"; RG 1.132; RG 1.206; and RG 1.208.

2.5.2.4 *Technical Evaluation*

The NRC staff reviewed Section 2.5.2 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of information presented in the FSAR and the DCD completely represents the required information related to vibratory ground motion. The staff's review confirmed that information contained in the application or incorporated by reference addresses the information required for this review topic. NUREG-1793 and its supplements document the results of the staff's evaluation of the information incorporated by reference into the LNP COL application.

The staff reviewed the following information in the LNP COL FSAR:

AP1000 COL Information Items

- LNP COL 2.5-2

The NRC staff reviewed LNP COL 2.5-2 related to COL Information Item 2.5-2 (COL Action Item 2.5.2-1), which addresses the provision for site-specific information related to the vibratory ground motion aspects of the site including: seismicity, geologic and tectonic characteristics, correlation of earthquake activity with seismic sources, PSHA, seismic wave transmission characteristics and the SSE ground motion. The COL information item in AP1000 DCD Section 2.5.2.1 states:

> Combined License applicants referencing the AP1000 certified design will address the following site-specific information related to the vibratory ground motion aspects of the site and region: (1) seismicity, (2) geologic and tectonic characteristics of site and region, (3) correlation of earthquake activity with seismic sources, (4) probabilistic seismic hazard analysis and controlling earthquakes, (5) seismic wave transmission characteristics of the site; and (6) SSE ground motion.

- LNP COL 2.5-3

The NRC staff reviewed LNP COL 2.5-3 related to COL Information Item 2.5-3 (COL Action Item 2.6-2), which addresses the provision for performing site-specific evaluations, if the site-specific GMRS at foundation level exceeds the response spectra in AP1000 DCD Figures 3.7.1-1 and 3.7.1-2 at any frequency, or if soil conditions are outside the range evaluated for the AP1000 DCD. The COL information item in AP1000 DCD Section 2.5.2.3 states:

> The Combined License applicant may identify site-specific features and parameters that are not clearly within the guidance provided in subsection 2.5.2.1. These features and parameters may be demonstrated to be acceptable by performing site-specific seismic analyses. If the site-specific spectra at foundation level at a hard rock site or at grade for other sites exceed the certified seismic design response spectra in Figures 3.7.1-1 and 3.7.1-2 at any frequency, or if soil conditions are outside the range evaluated for AP1000 design certification, a site-specific evaluation can be performed. These analyses may be either 2D or 3D. Results will be compared to the corresponding 2D or 3D generic analyses.

SER Section 2.5.2.4 provides the NRC staff's evaluation of the seismic, geologic, geophysical and geotechnical investigations carried out by the applicant to determine the site-specific GMRS, and the SSE ground motion for the site. The development of the GMRS is based upon a detailed evaluation of earthquake potential, taking into account the regional and local geology, Quaternary tectonics, seismicity, and site-specific geotechnical engineering characteristics of the site subsurface material.

During the early site investigation stage, the staff visited the site and interacted with the applicant regarding the geologic, seismic and geotechnical investigations conducted for the LNP COL application. To thoroughly evaluate the geologic, seismic and geophysical information the applicant presented, the staff obtained additional assistance from experts at the USGS. The staff, with its USGS advisors, made visits to the LNP Units 1 and 2 site in April and September 2009 (ML092600064 and ML093280825) to confirm interpretations, assumptions, and conclusions presented by the applicant related to potential geologic and seismic hazards. The staff's evaluation of the information the applicant presented in LNP COL FSAR Section 2.5.2 and of the applicant's responses to RAIs is presented below.

In addition to the RAIs addressing specific technical issues regarding vibratory ground motion at the LNP Units 1 and 2 site and discussed in detail below, the staff also prepared several editorial RAIs to clarify certain descriptive statements made by the applicant in the FSAR and to qualify FSAR figures and tables. These editorial RAIs are not discussed in this technical evaluation. Also, RAIs related to vibratory ground motion resolved in FSARs previously prepared for other sites in the CEUS are not discussed in detail in this technical evaluation for the LNP Units 1 and 2 site, but rather are addressed by a cross-reference to and a summary of the pertinent information used to satisfactorily resolve the issues as presented in those FSARs.

2.5.2.4.1 Seismicity

To characterize the seismic hazard for the LNP Units 1 and 2 site, the applicant followed the methodology provided in RG 1.208 and used the EPRI-SOG seismic hazard models

(EPRI-SOG, 1986), developed in the late1980s, as a starting point. The EPRI-SOG study used an earthquake catalog compiled through 1984 that covers the CEUS. FSAR Section 2.5.2.1 describes the applicant's update of the original EPRI-SOG earthquake catalog to extend it from 1985 through December 2006 and also to extend the coverage to include the portions of the Gulf of Mexico that were not covered in the original EPRI-SOG catalog.

2.5.2.4.1.1 EPRI-SOG Seismicity Catalog Updates

The staff focused its review of FSAR Section 2.5.2.1 on the adequacy of the applicant's description of the historical record of earthquakes. To update the EPRI-SOG earthquake catalog for the region surrounding the LNP Units 1 and 2 site, the applicant evaluated several different earthquake catalogs, including the ANSS, ISC, and NEIC catalogs.

2.5.2.4.1.2 Gulf of Mexico Seismicity

Because the EPRI-SOG earthquake catalog did not include events from the Gulf of Mexico except along its immediate coast, the applicant extended the coverage of its catalog to include seismicity within the Gulf of Mexico between latitude 24°N to 32°N and longitude 100°W to 83°W. The applicant's update was prompted in large part by two recent, moderate-magnitude seismic events in the Gulf. These events were the m_b 4.9 event that occurred on February 10, 2006, offshore of the Louisiana coast and the m_b 6.0 event that occurred on September 10, 2006, offshore of the Florida coast.

In FSAR Section 2.5.2.3, the applicant noted that due to the use of different magnitude conversion relationships its estimated m_b for the September 10, 2006 event, m_b 6.08, differs from that reported in the COL application submitted for STP Units 3 and 4, which gives m_b 6.11. In RAI 2.5.2-5, the staff asked the applicant to clarify the difference in the magnitude for the September 10, 2006, event as well as the different magnitude conversion relationships used in the STP Units 3 and 4 COL application (STPNOC, 2008) in comparison to those used in the LNP COL application. In response to RAI 2.5.2-5, the applicant explained that both LNP and STP averaged the output of three moment magnitude (M) and m_b relationships to calculate estimated m_b. Two relationships used are the same in both the LNP and STP COL applications. The third relationship differs. The applicant explained that STP used an earlier version of this relationship, while LNP utilized the final version of the conversion relationship. The staff reviewed the two different m_b estimates and finds that the difference in estimated m_b of 6.08 for LNP and 6.11 for STP is not significant. Both the 6.08 and 6.11 estimates are conservative since the value of directly measured m_b presented in the ANSS catalog is 5.8. Furthermore, the staff concludes that the slight difference in m_b for the September 10, 2006 Gulf earthquake does not affect the seismic hazard analysis at the LNP site. Therefore, the staff considers RAI 2.5.2-5 resolved.

2.5.2.4.1.3 Staff Conclusions Regarding Seismicity

Based upon its review of FSAR Section 2.5.2.1 and RAI 2.5.2-5, the staff concludes that the applicant developed a complete and accurate earthquake catalog for the region surrounding the LNP Units 1 and 2 site, including the Gulf of Mexico seismicity. The staff concludes that the seismicity catalog as described by the applicant in FSAR Section 2.5.2.1 forms an adequate basis for the seismic hazard characterization of the site and meets the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23.

2.5.2.4.2 Geologic and Tectonic Characteristics of the Site and Region

This SER section provides the staff's evaluation of the seismic source models the applicant used as part of its PSHA for the LNP Units 1 and 2 site. FSAR Section 2.5.2.2 describes the seismic sources from the original EPRI-SOG seismic source models (EPRI-SOG, 1986) that contribute to 99 percent of the total hazard at the CR3, located 15 km (9 mi) from the LNP Units 1 and 2 site. These seismic source models were developed in 1986 by the six EPRI-SOG ESTs. FSAR Section 2.5.2.4 describes the applicant's sensitivity studies to determine if the 1986 EPRI-SOG seismic source models needed updating based on more recent studies in the geologic and seismic literature cited by the applicant. Consistent with RG 1.208, the applicant evaluated more recent seismic hazard studies and data available for the region surrounding the site for comparison to the 1986 EPRI-SOG seismic source models. As a result of this evaluation, the applicant updated several of the original source models developed by the EPRI-SOG ESTs.

The staff's review of the application of the updated seismic source model to the hazard calculation at the LNP Units 1 and 2 site is discussed in SER Section 2.5.2.4.4.

2.5.2.4.2.1 Original EPRI-SOG Seismic Sources

In FSAR Section 2.5.2.4.3.1, the applicant describes its selection of EPRI-SOG seismic sources and stated that one relationship was used to convert m_b to M. Later in FSAR Section 2.5.2.4.2.3, the applicant presents three relationships used to convert m_b to M to use in its hazard analysis. In RAI 2.5.2-15, the staff asked the applicant to clarify why only one conversion relationship was used to select the EPRI-SOG seismic source zones and not three relationships, like those used in the hazard analysis. In response to RAI 2.5.2-15, the applicant explained that the hazard results are not very sensitive to the use of the alternative m_b to M relationships, as illustrated in FSAR Figure 2.5.2-236. Therefore, the applicant thought it was sufficient to use one relationship for the purpose of identifying the appropriate set of EPRI-SOG seismic sources. After reviewing the issues, the staff concludes that since the single relationship was used for the purpose of identification of EPRI-SOG sources only, and not for final hazard calculation where the applicant used the weighted average of the three formulas, this does not compromise the GMRS and hazard calculations. Therefore, the staff finds the applicant's response acceptable and considers RAI 2.5.2-15 resolved.

The three m_b to M conversion relationships the applicant presented in FSAR Section 2.5.2.4.2.3 are important to the applicant's hazard analyses because the magnitudes in the earthquake catalogs are in m_b whereas the ground motion prediction equations use M. However, the m_b scale saturates at m_b of 7, but the conversion relations go beyond m_b of 7. In RAI 2.5.2-19, the staff asked the applicant to clarify how it dealt with the issue of m_b saturation when performing magnitude conversion to use in its hazard analyses. In response to RAI 2.5.2-19, the applicant explained that the m_b to M conversion saturation has the most impact for the LNP Units 1 and 2 site in the characterization of the Charleston source, since that source is the only source affecting the LNP site with an m_b greater than 7. For the Charleston source zone, the repeated large earthquakes are initially characterized in terms of M. These M estimates were used directly to calculate the ground motions and hazard from this source, so that the m_b to M conversion was not necessary. Based on its review of the applicant's RAI response, the staff concludes that the saturation aspect of the m_b to M conversion relations has no material effect on the hazard analyses at the LNP site. This is due to the applicant's use of direct estimates of M for the Charleston source for which the m_b to M conversion relations were not used. The staff considers RAI 2.5.2-19 resolved.

2.5.2.4.2.2 Update of EPRI-SOG Seismic Source Models

FSAR Section 2.5.2.2.2 describes four PSHA studies that were completed after the 1989 EPRI PSHA and which involved the characterization of seismic sources within the LNP Units 1 and 2 site region. FSAR Sections 2.5.2.4.1 through 2.5.2.4.3 present the applicant's discussion and sensitivity analyses determining whether the 1986 EPRI-SOG seismic source models needed to be updated based on more recent seismic hazard studies or on new seismicity data for the region surrounding the LNP Units 1 and 2 site. The four PSHA studies that were completed after the 1989 EPRI PSHA include the USGS National Seismic Hazard Mapping Project (Frankel et al. 1996, 2002), the South Carolina Department of Transportation (SCDOT) seismic hazard mapping project (SCDOT, 2003), the LLNL Trial Implementation Program study (NUREG/CR-6607, Savy, et al., 2002), and the updated PSHA for the VEGP plant site (SNC, 2006). The applicant provided a description of these four models in FSAR Section 2.5.2.2.2, as well as a comparison of these more recent studies with the EPRI source PSHA models.

2.5.2.4.2.2.1 Update of the Charleston Seismic Source

The applicant updated the EPRI-SOG Charleston seismic source models with a model that was originally presented in the Site Safety Analysis Report (SSAR) for the VEGP ESP site (SNC, 2007). This update was based on the results of several post-EPRI PSHA studies (Frankel et al. 2002; Chapman and Talwani 2002) and the availability of paleoliquefaction data (Talwani and Schaeffer 2001). The applicant updated the EPRI characterization of the Charleston seismic source zone as part of the COL application. The applicant used the UCSS model to update the Charleston seismic source. The SSAR for the VEGP ESP Site (SNC, 2007) provides the details of the UCSS model and the SER for the VEGP ESP (NUREG-1923, 2009) describes the NRC staff's review of the UCSS. The UCSS model development followed the guidelines provided in RG 1.208 and used a Senior Seismic Hazard Analysis Committee (SSHAC) Level 2 (NUREG/CR-6372, "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and User of Experts") expert elicitation method to incorporate current literature cited by the applicant and data and the understanding of experts into an update of the Charleston seismic source model. The staff reviewed and approved the UCSS model as part of its review of the VEGP ESP application (NUREG-1923).

2.5.2.4.2.2.2 Gulf Coast Source Zones

Based on the geological and seismological data published since the 1986 EPRI-SOG seismic source model, the applicant evaluated whether the maximum magnitudes for the EPRI-SOG sources needed updating. As a result of two 2006 Gulf of Mexico earthquakes, the applicant updated the EPRI-SOG GCSZ.

The applicant updated five of the six EST GCSZ M_{max} distributions due to the occurrence of the February 10, 2006, m_b 4.9 earthquake and the September 10, 2006, m_b 6.0 earthquake in the Gulf of Mexico. The magnitudes of these two earthquakes exceeded, in some cases, the upper- and lower-bounds of the original EPRI GCSZ M_{max} distributions. To perform this update, the applicant implemented the GCSZ updates described in the STP Units 3 and 4 COL application (STPNOC, 2008). To determine what updates to make, STP performed a SSHAC Level 2 expert elicitation study (SSHAC, 1997). The purpose of the SSHAC process was to integrate expert opinion and to capture the center, body, and range of the scientific community's opinion on updating the EST GCSZ.

In RAI 2.5.2-16 and RAI 2.5.2-22, the staff asked the applicant to thoroughly describe the details of its GCSZ update, to provide further justification for the updated parameters, and to explain how the updated source models adequately characterize the seismic hazard of the Gulf Coast. In response, the applicant stated that not all the geometries of the six EST GCSZ encompass the locations of these two 2006 Gulf of Mexico earthquakes and therefore only the M_{max} distributions of sources that do encompass the earthquakes were updated. Additionally, the applicant proposed changes for a later version of FSAR Section 2.5.2, which describes in more detail the applicant's GCSZ update. For the reasons discussed in the following paragraphs, the staff finds these proposed FSAR changes acceptable. The staff is tracking these changes to the FSAR as **Confirmatory Item 2.5.2-1**.

<u>Resolution of Confirmatory Item 2.5.2-1</u>

Confirmatory Item 2.5.2-1 is an applicant commitment to update Section 2.5.2 of its FSAR. The staff verified that LNP COL FSAR Section 2.5.2 was appropriately updated. As a result, Confirmatory Item 2.5.2-1 is now closed.

STPNOC (2008) preformed a SSHAC Level 2 expert elicitation study (NUREG/CR-6372) to determine what updates to make to the GCSZ. The SSHAC Technical Integration (TI) team's original recommendation was for a M_{max} distribution with m_b and weights of 6.1 [0.1], 6.6 [0.4], 6.9 [0.4], and 7.2 [0.1]. However, the SSHAC Peer Review Panel (PRP) did not approve this M_{max} distribution. Instead, the SSHAC PRP recommended that the individual M_{max} distributions for five of the six ESTs GCSZ be updated. The applicant implemented the PRP M_{max} distribution in its update. As part of RAI 2.5.2-22, the staff asked the applicant to provide justification for not adopting the original TI team's M_{max} distribution. To address this, the applicant conducted a sensitivity analysis in which M_{max} distributions for the three GCSZ that encompass the September 10, 2006, earthquake were replaced with the TI team's original M_{max} distribution of: 6.1 [0.1], 6.6 [0.4], 6.9 [0.4], 7.2 [0.1]. SER Table 2.5.2-3 shows the resulting percent change in site ground motions at various spectral frequencies.

Table 2.5.2-3. Percent Change in LNP Site Ground Motions at Finished Grade Elevation Resulting from the Use of a M_{max} Distribution of 6.1 [0.1], 6.6 [0.4], 6.9 [0.4], 7.2 [0.1] (RAI 02.05.02-22 Table 1)

SPECTRUAL FREQUENCY (Hz)	PERCENT CHANGE IN LNP SITE GROUND MOTIONS AT FINISHED GRADE ELEVATION
0.5	+2
1.0	+4
2.5	+4
5.0	+6
10.0	+6
25.0	+7
100.0	+7

After reviewing the applicant's sensitivity study, the staff concludes that the updated M_{max} parameters adequately characterize the seismic hazard of the Gulf Coast region. The percent change results from the sensitivity analysis show that the higher M_{max} distribution originally recommended by the TI team does not greatly increase the seismic hazard at the LNP Units 1 and 2 site relative to the M_{max} distributions used by the applicant. Based on the modest size of the two 2006 Gulf earthquakes (m_b 4.9 and 6.0) and their distances from the site (758 and 498 km (471 mi and 309 mi)) the staff concludes that the applicants' updated GCSZ models adequately characterize the potential hazard.

RAIs 2.5.2-16 and 2.5.2-22 address the applicant's update of the EST GCSZ M_{max} distributions. The one source zone that the applicant did not update is the Woodward-Clyde Consultant background source model (WCC-B43). The staff asked the applicant to justify not updating that particular source and to describe how the source sufficiently characterizes the hazard for the Gulf. In response, the applicant described that the WCC-B43 background source is characterized by a 2-by-2 degree latitude-longitude zone centered near the LNP site and that neither recent Gulf of Mexico earthquake occurred within the zone. Additionally, the applicant compared the tectonic setting and type of crust of the WCC-B43 zone and that of the locations of the recent Gulf of Mexico earthquakes. The applicant demonstrated that the WCC-B43 zone primarily encloses the stable continental crust of the Florida platform, while the recent Gulf earthquakes occurred within transitional or oceanic crust (Johnston et al., 1994; Sawyer et al., 1991). The staff reviewed the tectonic and topographic maps the applicant provided in response to RAI 2.5.2-22 (RAI 2.5.2-22, Figures 2, 3, and 4). The staff concludes that, because of its placement and size about the LNP site, the WCC-B43 source zone was intended to characterize seismicity local to the site and not to characterize the entire Gulf Coast region. Additionally, the staff concludes from review of the tectonic and topographic maps that recent Gulf of Mexico earthquakes occurred in a type of crust different than the WCC-B43 zone characterizes. Finally, because the WCC-B43 zone characterizes seismicity locally about the LNP site in a crustal environment distinct from that of the recent Gulf events, the staff concludes that EPRI-SOG WCC-B43 background source model for the LNP Units 1 and 2 site does not need to be updated due to recent earthquakes in the Gulf of Mexico.

After reviewing the applicant's responses to RAIs 2.5.2-16 and 2.5.2-22, the staff concludes that the applicant justified its M_{max} parameters characterizing the seismic hazard of the Gulf Coast region, and that the EPRI-SOG WCC background source model for the LNP Units 1 and 2 site is not meant to characterize Gulf of Mexico seismicity and, therefore, does not need to be updated due to recent earthquakes in the Gulf of Mexico. The applicant's sensitivity study shows that the updated M_{max} parameters adequately characterize the seismic hazard of the Gulf Coast region. For these reasons, the staff considers RAIs 2.5.2-16 and 2.5.2-22 resolved.

2.5.2.4.2.2.3 Source Zones Outside of the Site Region

In accordance with RG 1.208, the applicant must expand the area of investigation beyond the site region if capable seismic source zones outside the site region are identified that produce large-magnitude earthquakes.

The New Madrid Seismic Zone (NMSZ), which extends from Missouri to Tennessee, is considered a major seismic zone in the CEUS. The NMSZ produced a series of large-magnitude earthquakes between December 1811 and February 1812. Paleoliquefaction studies in the region of the 1811-12 New Madrid earthquakes have identified several sequences of pre-historic earthquakes that have led researchers to estimate a mean recurrence interval for large NMSZ earthquakes of approximately 500 years. The applicant did not provide a discussion of the NMSZ's potential contribution to the seismic hazard at the LNP site. In RAI 2.5.2-18, the staff asked the applicant to discuss the significance of the NMSZ to the LNP Units 1 and 2 site and to provide justification for not including this source in the LNP PSHA.

In response to RAI 2.5.2-18, the applicant provided the staff with its evaluation results of the effect of NMSZ to the hazard at the LNP. SER Figure 2.5.2-6 compares the 2-second mean spectral acceleration hazard of repeated large-magnitude earthquakes for the NMSZ to that computed for earthquakes in the region of Charleston, South Carolina. The Charleston source was included in the LNP PSHA. SER Figure 2.5.2-6 illustrates that the mean hazard from the NMSZ is less than 1 percent of the hazard from the Charleston source for the 2-second spectral acceleration. The NMSZ is a distant source zone (> 1000 km (> 620 mi)) from the LNP Units 1 and 2 site. The effect of a large-magnitude earthquake on the site at such distances would be greatest at low frequencies, for example at 0.5 Hz equivalent to the 2-second period used by the applicant in SER Figure 2.5.2-6. Since the hazard of the NMSZ at the LNP Units 1 and 2 site is less than 1 percent of the Charleston source at low frequencies, the NMSZ contribution to the total hazard at the LNP Units 1 and 2 site will be less than that shown in SER Figure 2.5.2-6. Therefore, the NMSZ is not a significant contributor to the seismic hazard at the LNP Units 1 and 2 site. Based on the results of the applicant's testing of the NMSZ, the staff concludes that the NMSZ does not contribute significantly to the hazard at the LNP Units 1 and 2 site and, therefore, does not need to be included in the LNP PSHA. The staff considers RAI 2.5.2-18 resolved.

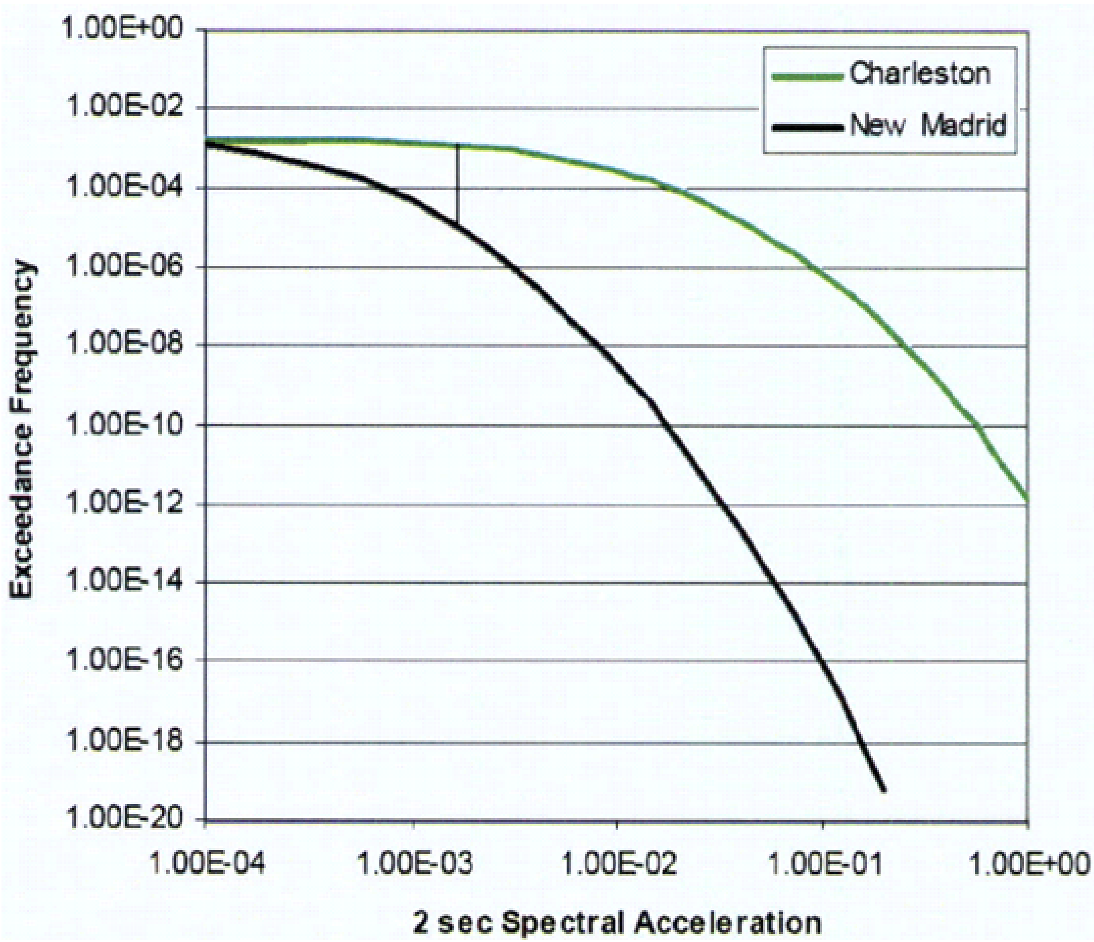


Figure 2.5.2-6. Mean Hazard Curves for the NMSZ (New Madrid) and Charleston Sources of Repeated Large-magnitude Earthquakes (RAI 02.05.02-18 Figure 1)

2.5.2.4.2.3 Staff Conclusions of the Geologic and Tectonic Characteristics of the Site and Region

Based upon its review of LNP COL FSAR Sections 2.5.2.2 and 2.5.2.4, the staff concludes that the applicant adequately updated the original EPRI-SOG seismic source models as the input to its PSHA for the LNP Units 1 and 2 site. In addition, the staff concludes that the applicant adequately considered seismic sources that were not part of the EPRI-SOG sources for the LNP Units 1 and 2 site, such as the NMSZ and the updated GCSZ. The staff concludes that the applicant's use of EPRI-SOG seismic source models in addition to the updates of the model, as described by the applicant in FSAR Section 2.5.2.2 and 2.5.2.4, forms an adequate basis for the seismic hazard characterization of the site and meets the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23.

2.5.2.4.3 Correlation of Earthquake Activity with Seismic Sources

FSAR Section 2.5.2.3 describes the correlation of updated seismicity with the EPRI-SOG seismic source model. The applicant compared the distribution of earthquake epicenters from both the original EPRI-SOG historical catalog (1627 to 1984) and the updated earthquake catalog (1985 to 2006) with the seismic sources characterized by the 1986 EPRI-SOG Project. The applicant concluded that there are no new earthquakes within the site region that can be associated with a known geologic structure and that there are no clusters of seismicity suggesting a new seismic source not captured by the EPRI-SOG seismic source model. The applicant also concluded that the updated catalog does not show a pattern of seismicity that would require significant revision to the geometry of any of the EPRI-SOG seismic sources.

In its review, the staff evaluated the completeness of the applicant's updated earthquake catalog and the applicant's subsequent conclusions by comparing the applicant's earthquake catalog to a compilation catalog derived from USGS seismicity catalogs. The USGS seismicity catalog from February 1985 to December 2006 is shown in SER Figure 2.5.2-7 as the yellow circles. The applicant's updated seismicity catalog is illustrated by the red circles, which covers February 1985 to December 2006. The comparison of these datasets illustrates that the applicant's updated earthquake catalog adequately characterizes the seismicity within and around the LNP Units 1 and 2 site region. The blue circles in SER Figure 2.5.2-7 illustrate the seismicity from the USGS catalog covering December 2006 to June 2010. This recent seismicity does not show any significant deviations from the applicant's seismicity catalogs. Based on the spatial distribution of earthquakes in the applicant's updated catalog and the staff's independent review of the USGS seismicity catalog through April 2010, the staff concludes that revisions to the existing EPRI-SOG source geometries are not warranted.

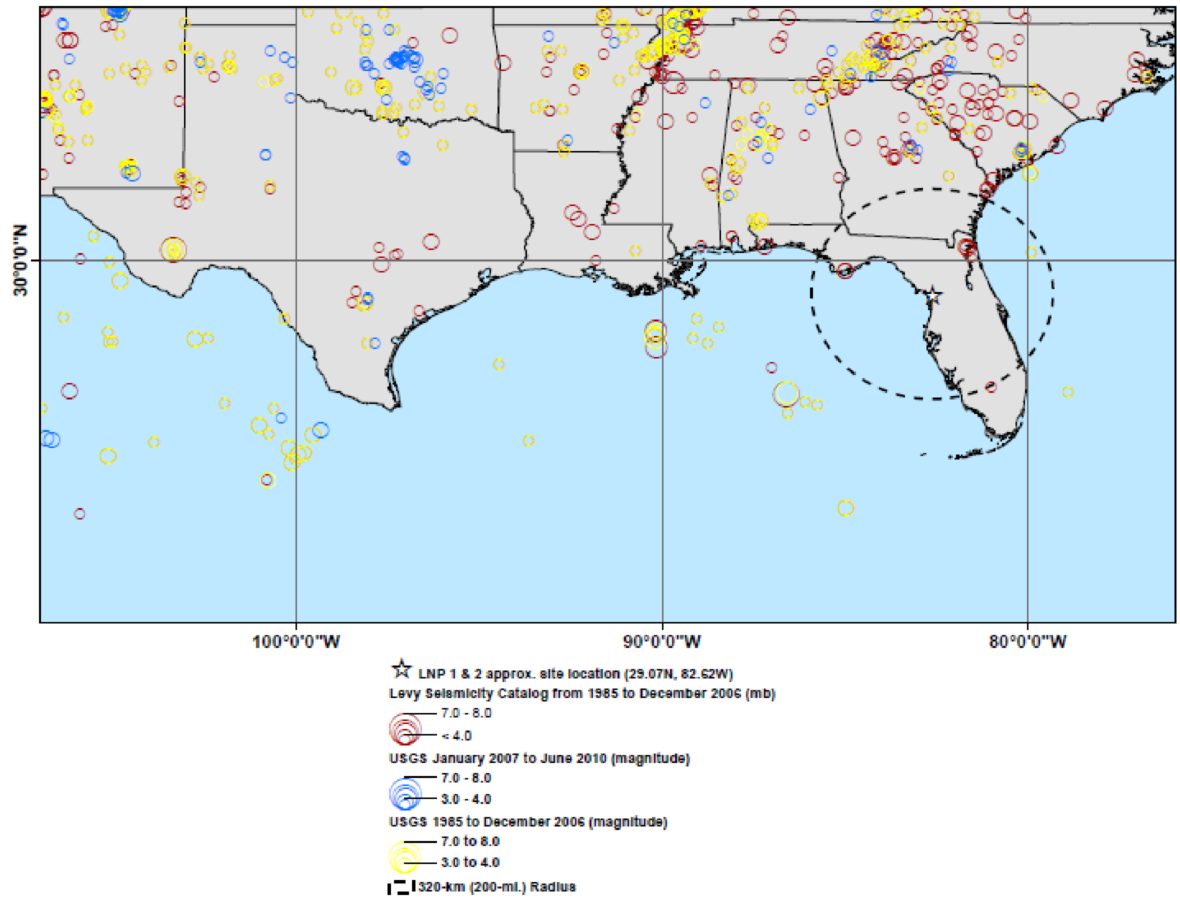


Figure 2.5.2-7. A Comparison of Events ($m_b \geq 3$) from the LNP Unit 1 and 2 Site Earthquake Catalog from 1985 to 2006 (Red Circles), the USGS Earthquake Catalog from 1985 to 2006 (Yellow Circles), and the USGS Earthquake Catalog from 2007 to 2010 (Blue Circles)

The star corresponds to the location of the LNP Unit 1 and 2 site and the dashed black oval corresponds to the 320-km (200-mi) site radius.

2.5.2.4.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

The staff focused its review of FSAR Section 2.5.2.4 on the applicant's updated PSHA and the LNP Units 1 and 2 site controlling earthquakes determined by the applicant after completion of its PSHA. The staff's review of the applicant's update of the EPRI-SOG seismic source model is described in SER Section 2.5.2.4.2, therefore this SER section focuses on the review of the application of the updated seismic source model to the hazard calculation at the LNP Units 1 and 2 site.

2.5.2.4.4.1 PSHA Calculation

The applicant performed PSHA calculations for PGA and spectral acceleration at frequencies of 0.5, 1.0, 2.5, 5, 10, 25, and 100 Hz. Following the guidance provided in RG 1.208, the PSHA calculations were performed assuming generic hard rock site conditions with V_S of 2.8 km/s (9,200 fps). The actual local site characteristics are incorporated in the calculation of the SSE spectrum, which uses the hard rock PSHA hazard results as the starting point.

2.5.2.4.4.2 Controlling and Deaggregation Earthquakes

FSAR Section 2.5.2.4.4.2 describes the deaggregation of final PSHA hazard curves to determine the controlling earthquakes for the LNP Units 1 and 2 site. To determine the LF and HF controlling earthquakes, the applicant followed the procedure outlined in RG 1.208. This procedure specifies that controlling earthquakes are determined from the deaggregation of the PSHA results corresponding to annual frequencies of 10^{-4}, 10^{-5}, and 10^{-6} and are based on the magnitude and distance values that contribute most to the hazard at the average of 1 and 2.5 Hz for LF and the average of 5 and 10 Hz for HF. The LF controlling earthquake for the site often represents a large distant source, while the HF controlling earthquake often corresponds to a smaller, local earthquake.

For the CR3 site, the HF controlling earthquake is M 5.3 at a distance of 17 km (10.5 mi). In RAI 2.5.2-7, the staff asked the applicant to explain the absence of a similar local, moderate-magnitude HF controlling earthquake for the LNP Units 1 and 2 site. In response to RAI 2.5.2-7, the applicant explained that the CR3 site seismic hazard analysis did not include the updated Charleston seismic source that produces large-magnitude earthquakes with a recurrence period of 500 years. Updating the Charleston source changed the contributions to the hazard, such that Charleston-type events are the major contributor to the HF hazard. The applicant also determined a weighted distribution of controlling earthquakes, which are called deaggregation earthquakes. As described in NUREG/CR-6728, deaggregation earthquakes separately address the contribution of nearby, intermediate, and distance events. SER Table 2.5.2-2 lists the LF and HF 10^{-4}, 10^{-5}, and 10^{-6} deaggregation earthquakes for the site and their associated weights. The deaggregation earthquakes include a nearby, or local, moderate-magnitude event as a contributor to the hazard. Since the Charleston source update resulted in changing the seismic source contributors, the staff concludes that a controlling earthquake similar to the CR3 site HF controlling earthquake [M 5.3, distance 17 km (10.5 mi)] is not necessary to characterize the hazard at the LNP Units 1 and 2 site. Additionally, the applicant’s calculation of deaggregation earthquakes, following the procedure outlined in Appendix D of RG 1.208, accurately determined the significant contributing events. Therefore, the staff concludes that the applicant adequately determined the LNP Units 1 and 2 site controlling and deaggregation earthquakes.

As described in FSAR Section 2.5.2.4.4.3, the applicant then used the updated ground motions discussed in SER Section 2.5.2.2.4, aleatory variability models, and the spectral shape functions of NUREG/CR-6728’s CEUS ground motions to develop response spectra to represent each of the controlling and deaggregation earthquakes. When assessing the uncertainty that arises due to inherent randomness in data, the aleatory variability, for the spectral frequencies between 0.1 and 100 Hz, the applicant used a combination of relationships from a number of references. In RAI 2.5.2-8, the staff asked the applicant to identify the sources of relations it used and to illustrate the dependence between the aleatory variability and frequencies that the applicant adopted. In its response, the applicant provided the justification of relations used as illustrated in SER Figure 2.5.2-8. The figure illustrates the relationship between aleatory variability (increase in Sigma) and frequency (or Period). The applicant provided the requested information; therefore, the staff considers RAI 2.5.2-8 resolved.

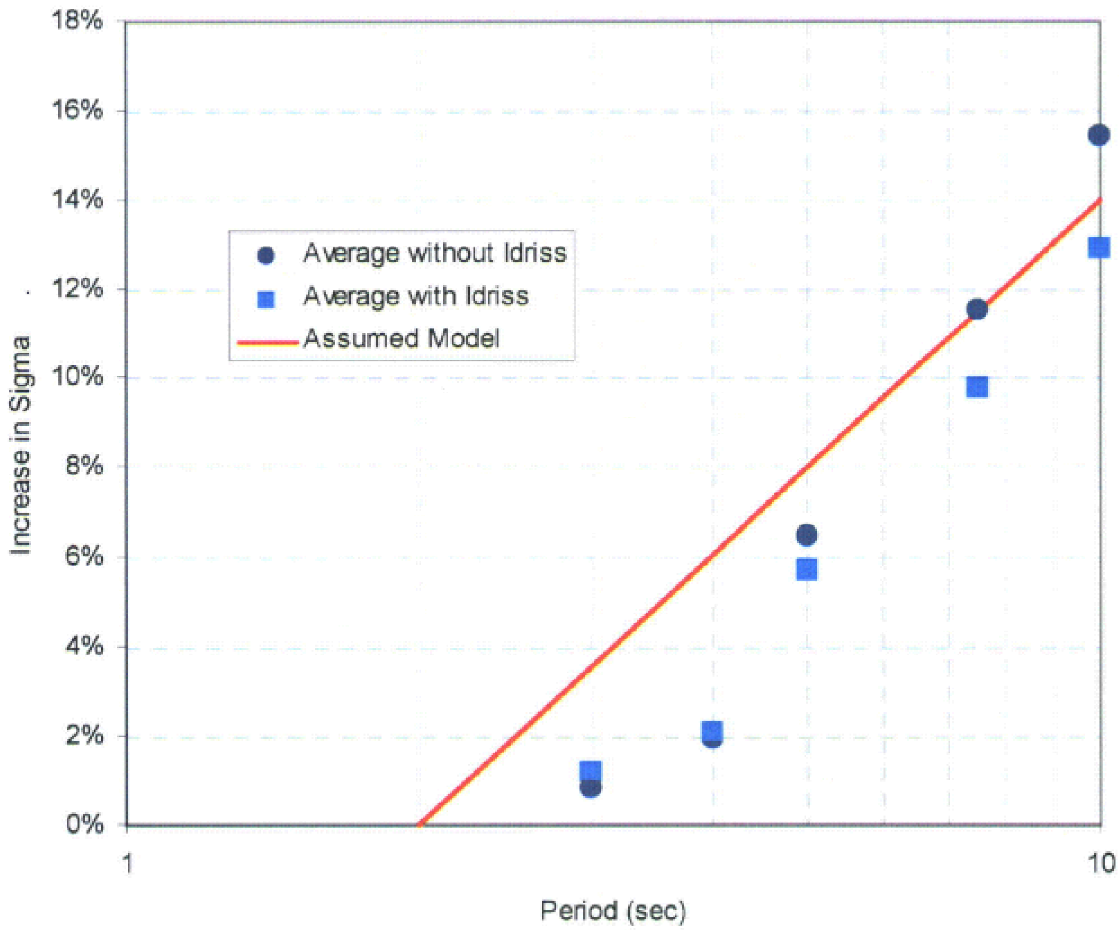


Figure 2.5.2-8. Increase in Aleatory Variability for Periods Longer than 2.0 Seconds Based on the PEER-NGA Ground Motions (RAI 02.05.02-08 Figure 1)

2.5.2.4.4.3 Staff Conclusions Regarding PSHA and Controlling and Deaggregation Earthquakes

After review of the applicant's PSHA and controlling and deaggregation earthquake determination and the applicant's responses to RAIs 2.5.2-7 and 2.5.2-8, the staff concludes that the applicant's PSHA adequately characterizes the seismic hazard for the region surrounding the LNP Units 1 and 2 site, that the controlling and deaggregation earthquakes determined by the applicant are representative of earthquakes that would be expected to contribute the most to the hazard and that the PSHA and controlling and deaggregation earthquakes determination meets the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23.

2.5.2.4.5 Seismic Wave Transmission Characteristics of the Site

FSAR Section 2.5.2.5 describes the method the applicant used to develop the LNP Units 1 and 2 site free-field ground motion spectra. The seismic hazard curves generated by the applicant's PSHA are defined for generic hard rock conditions (characterized by a V_S of 2.8 km/s (9,200 fps). According to the applicant, these hard rock conditions exist at a depth of 1,300 m (4,300 ft) beneath the ground surface at the LNP Units 1 and 2 site. To determine the site free-field ground motion, the applicant performed a site response analysis. The output of the applicant's site response analysis is the site amplification functions, which are used to determine the site-specific soil UHRS for the 10^{-4} and 10^{-5} hazard levels. To determine the soil UHRS, the applicant used Approach 2B outlined in NUREG/CR-6728. The 10^{-4} and 10^{-5} soil UHRS were then used to calculate the GMRS for the LNP Units 1 and 2 site.

2.5.2.4.5.1 Site Response Inputs

An important part of site response analysis is the model of the site subsurface soil and rock properties. Key properties include site stratigraphy, unit thickness, V_S and strain dependent behavior of each of the soil and rock layers underlying the site. The LNP Units 1 and 2 site location, within a 1 km (0.5 mi) radius, stratigraphy is known to a depth of approximately 1,370 m (4,500 ft) from oil exploration that took place around 1949. Stratigraphy to a depth of 150 m (500 ft) beneath the LNP Units 1 and 2 site is known from geotechnical borings that were drilled as part of the applicant's COL application study, which are described in FSAR Section 2.5.4.

2.5.2.4.5.1.1 Shear Wave Velocity

In FSAR Figure 2.5.2-249, the applicant shows four median V_S profiles for the shallow subsurface. In that figure, there are two LNP Unit 1 profiles and two LNP Unit 2 profiles, where the subsurface beneath each unit is described using both suspension logging data and downhole data. That figure demonstrates the differences in V_S measurements obtained by the two different methods. In RAI 2.5.2-11, the staff asked the applicant to clarify which of the two velocity measurements provide more reliable data and why. In response to RAI 2.5.2-11, the applicant explained that it did not assess which approach was more reliable and that it used all four profiles in the initial assessment of site amplification. The applicant then selected the site profiles, one for LNP Unit 1 and one for Unit 2 that produced the largest amount of amplification for use in the final site response analyses. Ultimately, the applicant enveloped the results of the two LNP sites to produce the final GMRS. The staff concludes that the procedure the applicant described is a conservative method to assess site amplification. Therefore, the staff finds the applicant's response adequate and considers RAI 2.5.2-11 resolved.

2.5.2.4.5.1.2 Density

In FSAR Section 2.5.2.5.1.3, the applicant discusses subsurface densities beneath the LNP Units 1 and 2 site. The applicant presented data in FSAR Section 2.5.4.2.3.2 for weathered and unweathered limestone showing densities increasing from 1.92 grams per cubic centimeter (g/cm^3; 120 pounds per cubic foot (pcf)) near the surface to 2.24 g/cm^3 (140 pcf) below elevation -91.5 m (-300 ft) msl. The applicant then described that V_S increase below the elevation of -305 m (-1,000 ft) msl and that it is likely that this velocity increase corresponds to an increase in density. Therefore, the applicant applied density of 2.4 g/cm^3 (150 pcf) for the rock layers below elevation -305 m (-1,000 ft) mean sea level (msl).

In RAI 2.5.2-20, the staff asked the applicant to provide a reference for a functional relationship between limestone velocity and density and then, based on that information; provide justification for the density of 2.4 g/cm^3 (150 pcf) at depths below -305 m (-1,000 ft.) msl. In response to RAI 2.5.2-20, the applicant provided a relationship between P-wave velocity and rock density for sedimentary rocks from Gardner et al. (1974). According to that relationship, a P-wave velocity of 3.66 km/s (12,000 fps) corresponds to a density of approximately 2.4 g/cm^3 (150 pcf). FSAR Figure 2.5.2-250 shows that P-wave velocities below -305 m (-1,000 ft) are in the range of 3.66 to 3.96 km/s (12,000 to 13,000 fps). Therefore, a density of 2.4 g/cm^3 (150 pcf) below -305 m (-1,000 ft) is consistent with Gardner's relationship. The staff concludes that the applicant provided sufficient justification for use of a density of 2.4 g/cm^3 (150 pcf) below -305 m (-1,000 ft) at the LNP Units 1 and 2 site. The staff considers RAI 2.5.2-20 resolved.

2.5.2.4.5.1.3 Karst Feature Characterization and Permeation Grouting Program

In order to understand how thoroughly the subsurface karst features were characterized by geophysical testing and the extent of the applicant's grouting program, in RAI 2.5.2-2, the staff asked the applicant why geophysical tools, such as resistivity, microgravity, and seismic tomography were not used to further characterize the extent of subsurface karst features.

In response to RAI 2.5.2-2, the applicant described that during pre-COL application site selection investigations, surface refraction and microgravity surface geophysical surveys were performed in addition to a series of preliminary boreholes. The applicant found that these investigation methods did not produce reliable results at the site due to subsurface heterogeneities. As a result, the COL application investigation instead included a large number of borehole geophysical loggings and surveys. Seismic tomography was tested at the Savannah River Site in an attempt to characterize "soft zones" at a depth of approximately 44 m (145 ft). The staff reviewed the Savannah River Site Report (Cumbest, et al., 1996). In the report, seismic tomography discerned anomalous layers, but identification of specific cavities, including karst features, was not successful. Also, microgravity and electrical resistivity are insufficiently sensitive to characterize such features and the reliability of these technologies to find subsurface karst features is estimated as poor or fair. Regarding the geophysical tools the applicant used to characterize the extent of potential subsurface karst features, the staff concludes that additional geophysical investigations would not improve characterization of the site's subsurface karst features and that the applicant used adequate methods to characterize the extent of subsurface karst features. The staff considers RAI 2.5.2-2 resolved.

In RAI 2.5.2-1, the staff asked the applicant to describe its plans for ensuring that V_S post-grouting at the site was appropriately represented in the site response analyses. Since the applicant's permeation grouting program will inject grout material permanently into the subsurface beneath the LNP Units 1 and 2 site, in this RAI, the staff questioned whether the applicant's site characterization, including site uniformity and V_S, presented in its COL application will remain accurate after grout injection.

To address the staff's concerns, the applicant conducted a grout test program. The purpose of the grout test program was to validate the applicant's permeation grout program design and grouting techniques, to measure the change in V_S and permeability of the grouted zone, and to determine the amount of grout take in the subsurface. The applicant presented the shear wave test results from its grout test program. During the grout test program, the applicant made pre- and post-grouting measurements of V_S using P-S suspension logging. SER Figure 2.5.2-9 shows the applicant's seismic wave velocity results for pre- and post-grouting measurements. The pre- and post-grouting measurements were performed in cased 10-cm (4-inches (in)) borehole PVC pipe. The applicant additionally addressed a concern of the staff regarding this P-S suspension logging methodology. The staff's concern was whether the casing surrounding the borehole piping affected the applicant's velocity measurements. In response, the applicant provided a figure, which is now SER Figure 2.5.2-10, that illustrates a comparison between velocities obtained using the cased 10-cm (4-in) borehole PVC pipe P-S suspension logging methodology and the downhole layered model methodology.

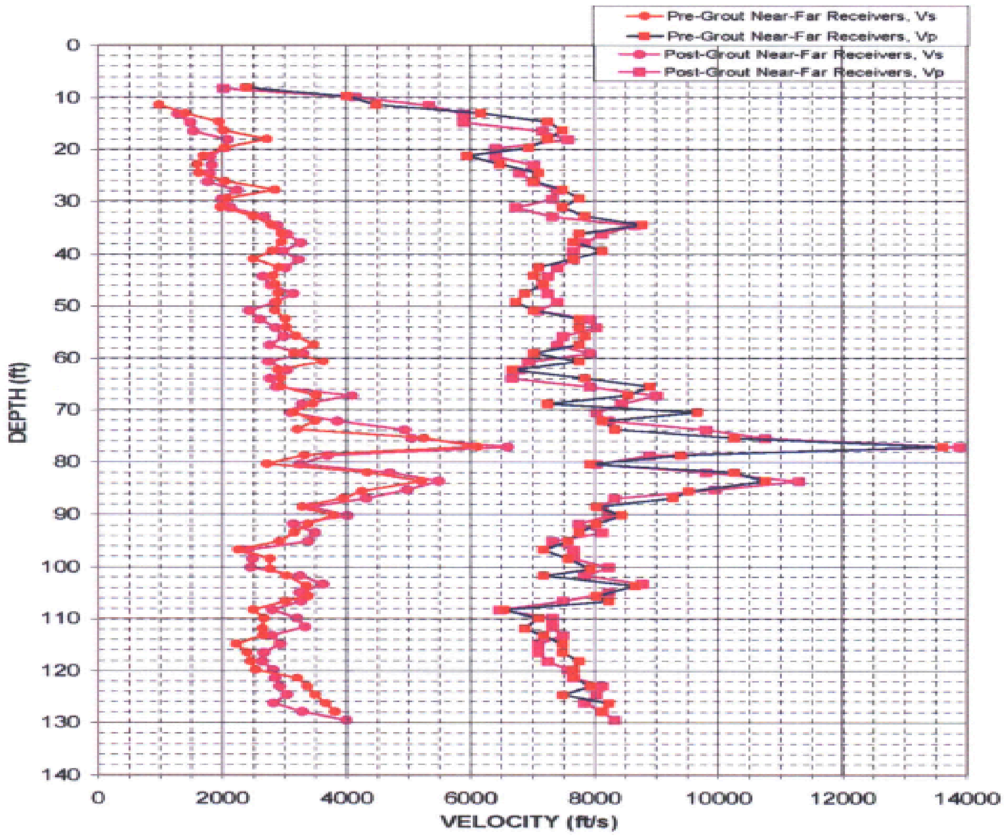


Figure 2.5.2-9. Pre- and Post-grouting Compressional (V_p) and Shear Wave (V_s) Velocities Suspension Logging Measurements (RAI Figure 02.05.02-1-01)

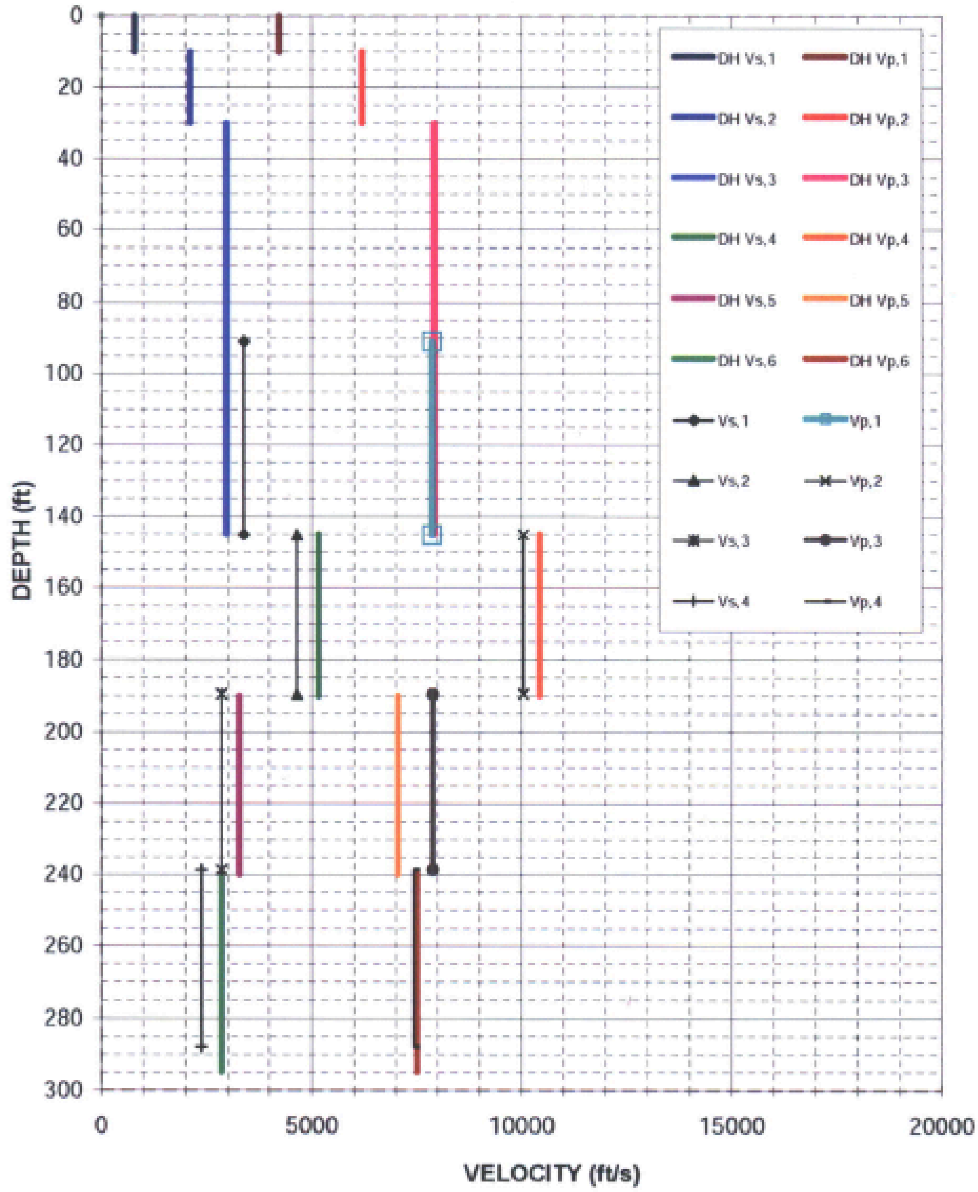


Figure 2.5.2-10. Seismic Wave Velocities Measured Using the Cased P-S Suspension Logging Methodology (Black Lines and Symbols) and Using Downhole Layered Models (Line and Symbols Labeled DH) (RAI Figure 02.05.02-1-02)

Regarding the pre- and post-grouting seismic wave velocities, the staff concludes that after the permeation grouting program is concluded, the LNP Units 1 and 2 site will maintain its site uniformity and V_S characterization as described in the LNP Units 1 and 2 COL application. As shown in SER Figure 2.5.2-9, the pre- and post-grouting measurements are within the expected precision of the P-S suspension logging method, and the change in V_S from pre- to post-grouting is within the standard deviation for the upper layers of the Avon Park Formation. Additionally, SER Figure 2.5.2-10 shows that both the cased and uncased P-S suspension logging methods produce similar seismic velocities versus depth and the comparison illustrates that the casing used in the borehole measurements did not systematically affect seismic wave velocity measurements during P-S suspension logging data collection. These comparative results of the cased P-S suspension logging and downhole layered models, assure the staff that the cased borehole piping did not significantly affect the applicant's seismic wave measurements. The staff considers RAIs 2.5.2-1 resolved.

2.5.2.4.5.1.4 Acceleration Time Histories

In FSAR Section 2.5.2.5.2, the applicant discusses its use of acceleration time histories for input rock motions in the site response analysis. The applicant developed response spectra for each controlling and deaggregation earthquake for the HF and LF ranges and 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. Thirty time histories were chosen from the sets given in NUREG/CR-6728. The applicant scaled the time histories to match the target earthquake spectrum. However, the applicant did not provide the specific information about which acceleration time histories it chose from NUREG/CR-6728 and it provided minimal description of the scaling procedure used to match the spectra to the target earthquake spectra. In RAI 2.5.2-12, the staff asked the applicant to provide a list of the actual time histories used, specifically describing earthquakes and stations, which recorded the motion, and to describe in detail how the records were scaled.

In response to RAI 2.5.2-12, the applicant provided a list of the specific recordings used and recording parameters such as date, time, magnitude, station, and distance from event to station, among others. The applicant chose recordings from active tectonic regions and modified the spectra to have the general characteristics expected for rock site motions in the CEUS. Regarding the scaling of the input response spectra, the applicant explained that it first defined a target spectrum for each controlling and deaggregation earthquake. Second, the applicant scaled the individual input acceleration time histories in the frequency domain to match the target spectra. SER Figure 2.5.2-11 shows the time history from NUREG/CR-6728 and its response spectrum, the target spectrum, and the scaled time history and its response spectrum. After reviewing the specific list of the acceleration time histories used as input motions the staff concludes that the applicant demonstrated appropriate use of the input acceleration time histories and of the scaling process, because the applicant used inputs and scaling consistent with the controlling and deaggregation earthquakes. The staff finds the applicant's response to RAI 2.5.2-12 adequate and considers this RAI 2.5.2-12 resolved.

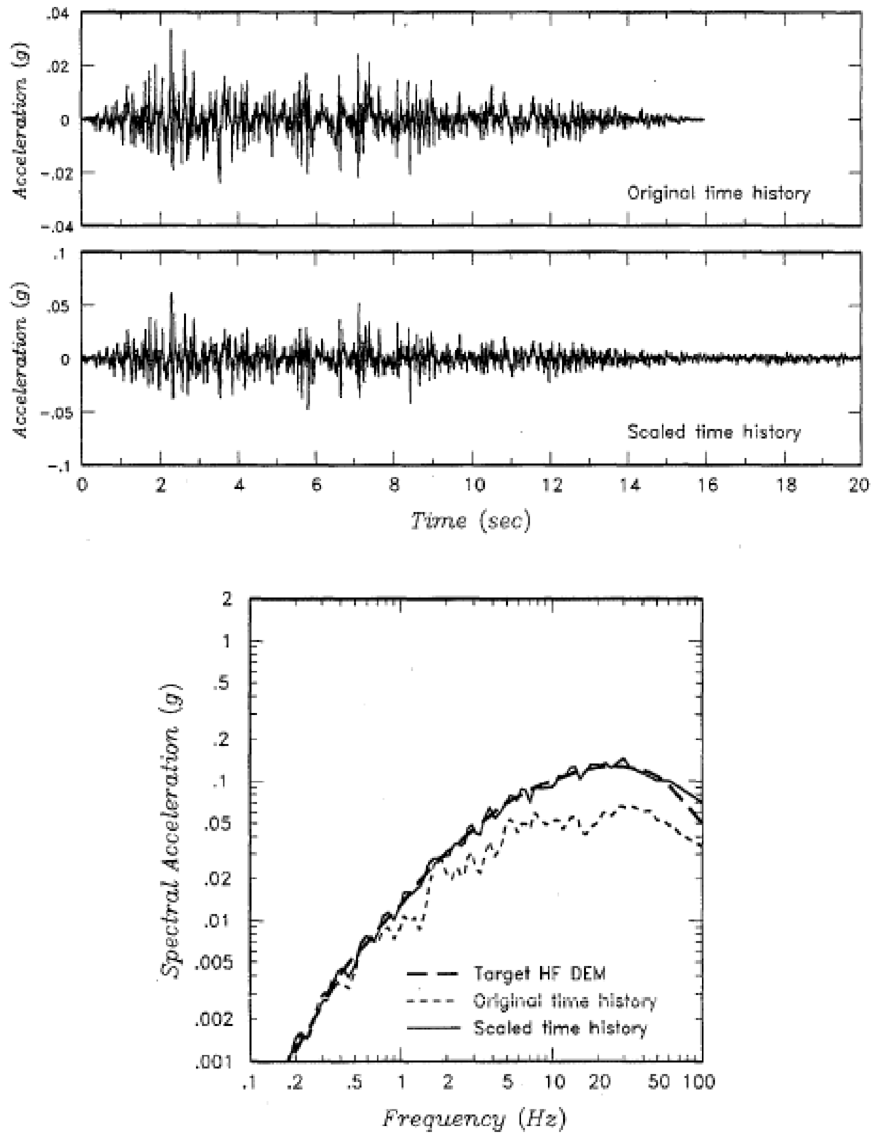


Figure 2.5.2-11. An Example of the Applicant's Scaling of Input Acceleration Time Histories Using the Parkfield Earthquake, San Luis Obispo 234° Component to Match the 10^{-4}, HF, DEM Earthquake Target Spectrum (RAI 02.05.02-12 Figure 2)

2.5.2.4.5.2 Site Response Methodology

In FSAR Section 2.5.2.5, the applicant describes the methodology it used to develop the soil UHRS for the 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. To determine the soil UHRS, the applicant used Approach 2B outlined in NUREG/CR-6728, in which the applicant first developed soil models;

next randomized the soil profiles to account for variability; and lastly performed the final site response analysis.

FSAR Section 2.5.2.5.1 discusses the dynamic properties of the LNP Units 1 and 2 site. Seismological methods of site response calculations, including Approach 2B and analyses using the one-dimensional SHAKE program (Schnabel et al., 1972), used by the applicant are based on the assumption of a uniform (flat) layer structure under the site. In RAI 2.5.2-3 and RAI 2.5.2-21, the staff asked the applicant to justify the assumption of uniformity of layers based on the available boring and shear wave profiles, to clarify how variability was accounted for in the site response analysis, and to justify the use of only one V_S base model.

In response, the applicant described that analysis results indicate rock layer dips of 1 to 2 degrees and velocity data from three deep wells illustrate similar trends at depth. Likewise, the top of the basement rock dips at about 1 degree. To address variability in V_S, the applicant constructed four initial base case velocity profiles, calculated individual site responses for each, and chose the two profiles, one for LNP Unit 1 and one for LNP Unit 2 that resulted in the largest site amplification. The two chosen amplification functions were used to develop a single GMRS for the LNP site.

In order to review the applicant's responses to RAI 2.5.2-3 and 2.5.2-21, the staff evaluated the results of the dip analysis of the rock strata, the velocity data from the three deep wells, and the data regarding dip of the top of the basement rock. Dip analysis and well data indicate that the strata are flat-lying and suitable for use in the one-dimensional site response analysis. Additionally, the variability in layer velocity is accounted for by the use of multiple base-case profiles and then enveloping the site response amplification functions. For these reasons, the staff concludes that the assumption of a uniform (flat) layer structure under the LNP site is appropriate for the applicant to use for its site response analysis. In addition, the staff concludes that the applicant conservatively accounted for variability in V_S by enveloping the site amplification functions. The applicant provided sufficient information to address the staff's RAIs and the staff considers RAIs 2.5.2-3 and 2.5.2-21 resolved.

2.5.2.4.5.3 NRC Site Response Confirmatory Analysis

To determine the adequacy of the applicant's site response calculations, the staff performed its own confirmatory site response analysis. As input to its calculations of GMRS, the staff used the static and dynamic soil properties provided in FSAR Table 2.5.2-222 for LNP Unit 1 and FSAR Table 2.5.2-223 for LNP Unit 2. Those profiles consist of 29 layers on the top of hard rock at the depth of 1,325 m (4,350 ft) for GMRS at the elevation of 11 m (36 ft) NAVD88. The staff performed the site response calculations using the programs SHAKE2000 and STRATA, which are both based on the equivalent linear (EQL) method. To represent the input motions, the staff used 17 time histories of earthquakes similar in size and source-to-site distances to that of controlling earthquakes shown in SER Table 2.5.2-2. The staff weakly scaled the time histories. The staff first calculated site amplification functions for each of the 29-layer V_S profiles of LNP Units 1 and 2. Next, the staff took the maximum of the two site amplifications. Lastly, the staff enveloped the maximum of the two LNP Units 1 and 2 site amplification functions. The staff's resulting amplification curves are compared with the applicant's GRMS amplification functions in SER Figure 2.5.2-12. In the frequency range 0.1 to 30 Hz and 80 to 100 Hz (PGA), the applicant's site amplification functions are equal or exceed the staff's site amplification. The staff's site amplification function exceeds the applicant's in the frequency range of 30 to 75 Hz. This exceedance is not significant because of the limitations of methods used, where the EQL method produces accurate results up to the frequencies of 25 Hz. Furthermore, GMRS calculated using this AF is still much lower than the CSDRS. Therefore, the staff concludes that in the frequency range significant to a reactor's structures, systems, and components, there are no significant differences between the staff's and the applicant's calculated amplification functions for the LF and HF, 10^{-4} and 10^{-5} hazard levels.

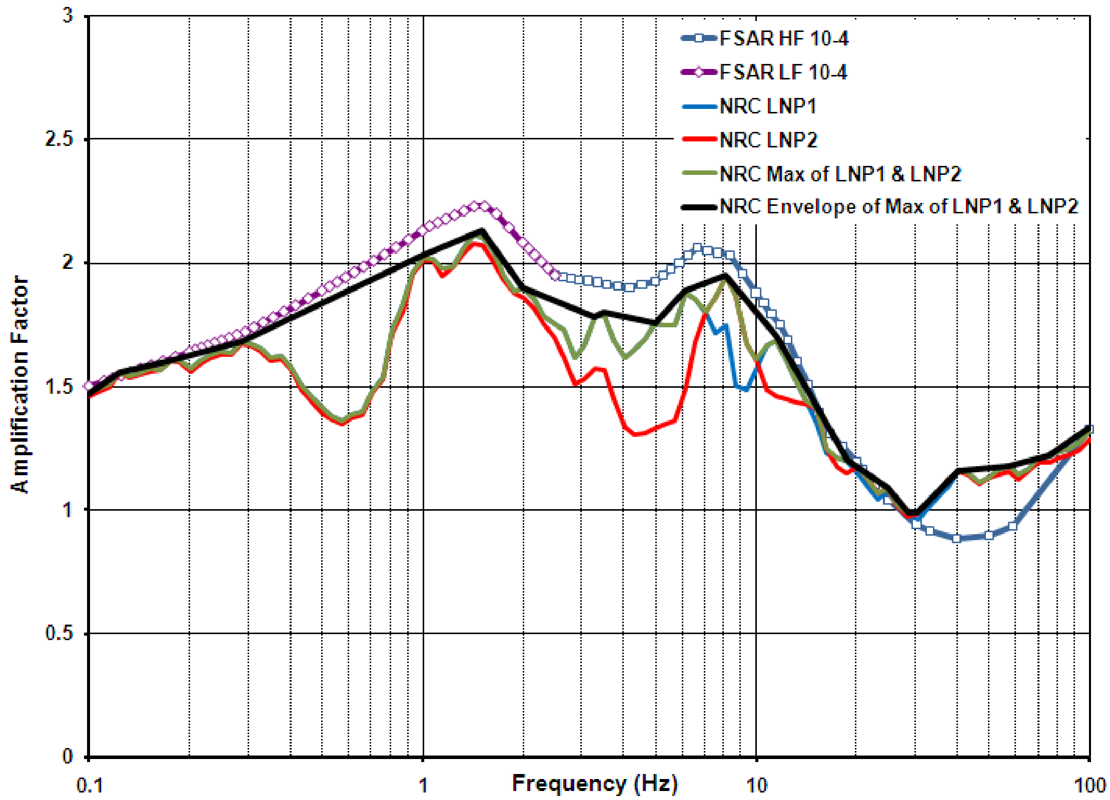


Figure 2.5.2-12. Results of the Staff's Confirmatory Analysis

2.5.2.4.5.4 Staff Conclusions Regarding Seismic Wave Transmission Characteristics of the Site

Based on the results of the staff's confirmatory analysis and the applicant's responses to RAIs 2.5.2-1 through 2.5.2-3, RAI 2.5.2-11, RAI 2.5.2-12, RAI 2.5.2-20, and RAI 2.5.2-21 discussed above, the staff concludes that the applicant's site response inputs, methodology, and results are acceptable. Specifically, the staff concludes that the applicant's site response inputs adequately characterize the site subsurface, that the permeation grouting program will not alter the site uniformity or V_S structure at the site, and that applicant adequately accounted for variability in V_S by enveloping the site amplification functions. The applicant used appropriate approaches to incorporate soil property uncertainties and followed the guidance provided in RG 1.208, which meets the requirements of 10 CFR 52.79(a)(1)(iii) and 10 CFR 100.23. This conclusion is further supported by the results of the confirmatory site response calculations performed by the staff that are similar to the applicant's results.

2.5.2.4.6 Ground Motion Response Spectra

RG 1.208 defines the GMRS as the site-specific SSE to distinguish it from the certified seismic design response spectra (CSDRS), the design ground motion for the AP1000 certified design. FSAR Section 2.5.2.6 describes the method the applicant used to develop the horizontal and vertical, site-specific GRMS. To obtain the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and ASCE/SEI Standard 43-05 and additionally multipled the spectrum by a 1.212 scale factor. To develop the vertical GMRS, the applicant used V/H ratios, based on NUREG/CR-6728. The applicant's horizontal and vertical GMRS are shown in SER Figure 2.5.2-5.

In FSAR Section 2.5.2.6.4, the applicant describes its development of the vertical GMRS. The applicant used NUREG/CR-6728 to develop V/H ratios for an intermediate site, where an intermediate site has a subsurface characterized between rocks typical of sites in the WUS and sites in the CEUS. In RAI 2.5.2-13, the staff asked the applicant to clarify why the LNP Units 1 and 2 site was considered as intermediate and to justify the value used for kappa. In response to RAI 2.5.2-13, the applicant explained the kappa site value of 0.022 seconds was calculated using the EPRI (2005) empirical relationship between kappa and V_S. The applicant's kappa value of 0.022 seconds is between the typical value assigned to the WUS rock sites (0.04) and the value used for CEUS (0.006). The applicant stated that based on this kappa value, the peak in the V/H response spectral ratio would be expected to occur at an intermediate frequency between the values near 15 and 63 Hz for WUS and CEUS. Both EPRI (2005) and NUREG/CR-6728 are documents that the NRC supports for the use of seismic hazard analyses. Since the applicant developed its V/H ratios using these documents and the applicant's implementation of these documents was consistent with characterizing the site as intermediate, the staff concludes that the LNP Units 1 and 2 site is appropriately characterized as an intermediate site. The staff concludes that the applicant's calculated kappa values and V/H ratios for the LNP Units 1 and 2 site are acceptable. The staff considers RAI 2.5.2-13 resolved.

Based on the applicant's use of the standard procedure outlined in RG 1.208 to develop both the horizontal and vertical GMRS and the applicant having increased those spectra by a scale factor of 1.212, as well as on the applicant's responses to RAI 2.5.2-13, the staff concludes that the applicant's GMRS adequately represents the LNP Units 1 and 2 site ground motion.

2.5.2.4.7 Sensitivity Study of CEUS Seismic Source Characterization Model

On March 15, 2012, the NRC sent RAI Letter No. 108 (Agencywide Document Access and Management System (ADAMS) No. ML120550146) to the applicant. That letter explained that the staff was implementing some of the Fukushima Near-Term Task Force recommendations, as described in SECY-12-0025, "Proposed Orders and Requests for Information in Response to

Lessons Learned from Japan's March 11, 2011, Great Tohoku Earthquake and Tsunami" (ADAMS Accession No. ML12039A111). Among other topics, RAI Letter No. 108 requested that the applicant evaluate seismic hazards at the LNP site against current NRC requirements and guidance as described in SECY-2012-0025 Enclosure 7, Attachment 1 to Seismic Enclosure 1 (ADAMS Accession No. ML12039A188), and, if necessary, update the design basis and structures systems and components important to safety to protect against the updated hazards. The applicant responded to RAI Letter No. 108 in Progress Energy Letter NPD-NRC-2012-029 (ADAMS Accession No. ML122230155). The staff's evaluation of the applicant's response is located in SER Section 20.1. Based on the evaluation, the staff concludes that the scaled site-specific ground motions developed using the updated EPRI-SOG model with the CAV filter presented in FSAR Section 2.5.2.6 are appropriate for use as the design basis for the LNP site.

2.5.2.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.5.2.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff's review confirmed that the applicant addressed the required information relating to vibratory ground motion, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the staff reviewed the seismic information submitted by the applicant in LNP COL FSAR Section 2.5.2. On the basis of its review of the information in LNP COL 2.5-2 and LNP COL 2.5-3, the staff finds that the applicant has provided a thorough characterization of the seismic sources surrounding the site, as required by 10 CFR 100.23. In addition, the staff finds that the applicant adequately addressed the uncertainties inherent in the characterization of these seismic sources through a PSHA, and this PSHA follows the guidance provided in RG 1.208. The staff concludes that the controlling earthquakes and associated ground motion derived from the applicant's PSHA are consistent with the seismogenic region surrounding the LNP Units 1 and 2 COL site. In addition, the staff finds that the applicant's GMRS, which was developed using the performance-based approach, adequately represents the regional and local seismic hazards and accurately includes the effects of the local site subsurface properties. The staff concludes that the proposed LNP Units 1 and 2 COL site is acceptable from a geologic and seismologic standpoint and meets the requirements of 10 CFR 52.79 (a)(1)(iii) and 10 CFR 100.23.

2.5.3 Surface Faulting

2.5.3.1 *Introduction*

LNP COL FSAR Section 2.5.3 discusses the potential for tectonic (i.e., due to faulting) and non-tectonic surface and near-surface deformation at the LNP site. The applicant collected information related to both tectonic and non-tectonic surface and near-surface deformation during the LNP site characterization investigations and presented this information in the LNP COL FSAR in regard to the following specific topics: geologic, seismic, and geophysical investigations; geologic evidence, or absence of evidence, for surface deformation, including lineament analysis; correlation of earthquakes with capable tectonic sources; ages of most recent deformations; relationship of tectonic structures in the site area to regional tectonic structures; characterization of capable tectonic sources; designation of zones of Quaternary (2.6 Ma to present) deformation in the site region; and potential for tectonic and non-tectonic surface deformation at the site, including that associated with karst development.

2.5.3.2 *Summary of Application*

Section 2.5 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.5.3 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.5.3, the applicant provided site-specific information to address the following:

<u>*AP1000 COL Information Item*</u>

- LNP COL 2.5-4

The applicant provided additional information in LNP COL 2.5-4 to address COL Information Item 2.5-4 (COL Action Item 2.5.3-1). LNP COL 2.5.4 addresses the evaluation of site-specific subsurface geologic, seismic, and geophysical information in regard to the potential for surface or near-surface faulting at the site.

The applicant developed LNP COL FSAR Section 2.5.3 for the LNP site based on information derived from review of existing geologic and seismicity data and published literature; discussions with experts in geology, seismology, tectonics, and karst development who have conducted recent research in and around the site area; geologic field reconnaissance studies in the site vicinity and site area and at the site location; lineament analyses using aerial photographs and remote sensing imagery; and detailed investigations performed for the LNP COL application, including subsurface borings, surface geophysical testing, and downhole geophysical logging and seismic testing. The applicant also incorporated limited information applicable to the LNP site based on the CR3 FSAR (Florida Power Corporation, 1976), particularly in regard to fracture orientations and a lack of data indicative of faulting. The CR3 site is located about 18 km (11 mi) southwest of the LNP site.

Based on the information sources defined above, the applicant concluded in FSAR Section 2.5.3 that no deformational or geomorphic features indicative of potential Quaternary (2.6 Ma to present) tectonic activity at the LNP site have been reported in the literature, and that none were identified either by the site area experts or during the detailed field investigations performed for the LNP COL application. Following SER Sections 2.5.3.2.1 through 2.5.3.2.8 present a summary of the information provided by the applicant in LNP COL FSAR Section 2.5.3 related to tectonic surface deformation due to faulting, as well as non-tectonic surface deformation.

2.5.3.2.1 Geologic, Seismic, and Geophysical Investigations

FSAR Section 2.5.3.1 briefly describes the geologic, seismic, and geophysical investigations the applicant performed at the LNP site and in the site area to evaluate the potential for tectonic surface deformation, including surface fault rupture. The applicant cross-referenced FSAR Sections 2.5.1.2.1.3 and 2.5.1.2.4, which include detailed summaries of the information the applicant used to evaluate karst and site area and site vicinity structural geology, respectively, and concluded that no documented Quaternary (2.6 Ma to present) faults occur within the site region, site vicinity, or site area and that no capable tectonic sources exist therein. The applicant extended this conclusion to the faults postulated by Vernon (1951) to occur within the site vicinity and site area, which were also discussed in the FSAR for the CR3 site (Florida Power Corporation, 1976), based on the fact that no well-documented geologic evidence exists for these faults. The applicant also discussed the faults proposed by Vernon (1951) in FSAR Section 2.5.3.2 as addressed below in SER Section 2.5.3.2.2.

2.5.3.2.2 Geologic Evidence, or Absence of Evidence, for Surface Deformation

FSAR Section 2.5.3.2 discusses the presence or absence of surface deformation within the LNP site area. The applicant stated that recent geologic maps and evaluations of subsurface data do not show any structural features within the LNP site area. However, the applicant indicated that Vernon (1951) postulated seven faults in Citrus and Levy counties, three of which lie within the site area. These three postulated faults, the Inverness fault and two unnamed faults designated as Faults “A” and “B”, are shown in SER Figure 2.5.3-1 (reproduced from FSAR Figure 2.5.3-201). The applicant indicated that the northern end of the postulated Inverness Fault is located approximately 2 km (1.2 mi) east of the LNP site, and postulated Faults A and B are located about 4 km (2.5 mi) southwest and 7 km (4.3 mi) northeast of the site, respectively.

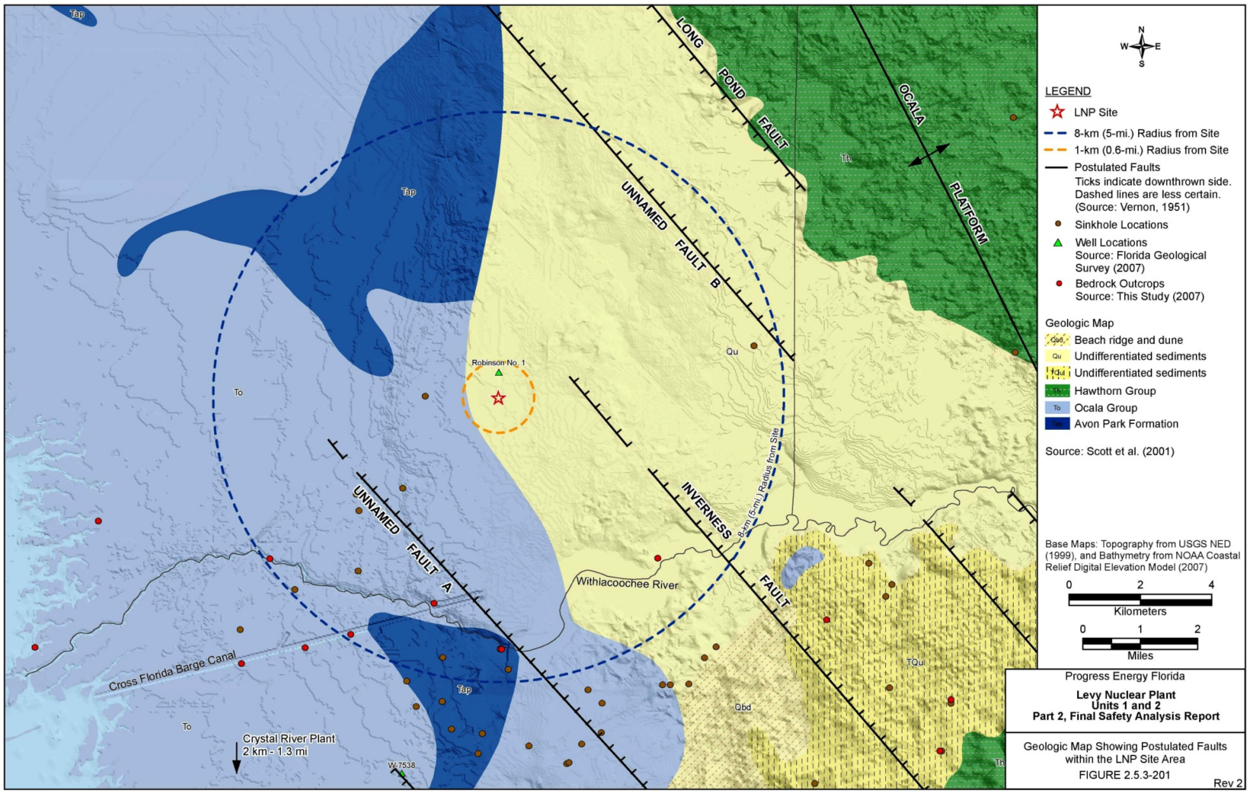


Figure 2.5.3-1. Geologic Map Showing Faults Postulated by Vernon (1951) to Lie Within the LNP Site Area (FSAR Figure 2.5.3-201)

The applicant reported that the faults postulated by Vernon (1951) to occur in the site area, based on his analysis of lineaments and interpretation of sparse geologic data, could not be detected in aerial photographs acquired in 1949; in Landsat images acquired in 2000; in a 10 m (32.8 ft) resolution USGS DEM; or in a DEM developed from 2007 light detection and ranging (LIDAR) data. The applicant also noted that the postulated faults do not disrupt marine terraces in the site area, which are estimated to be Late to Early Pleistocene (2.6 to 0.01 Ma), or possibly Pliocene (5.3 to 2.6 Ma), in age. The applicant further indicated that stratigraphic units used by Vernon (1951) to postulate the faults could not be differentiated, and that he based his interpretations on inferred correlation of stratigraphic units between widely-spaced outcrops and borehole data such that identification of the faults was highly speculative. The applicant cited Scott (1997), who noted that, because Vernon (1951) identified many of his faults based on interpreted offsets of the top of the Ocala Limestone, a surface with as much as 50 m (164 ft) of relief due to karst development, identification of faulting would be difficult at best.

Based on the results of research and geologic mapping as stated above, which post-date the work of Vernon (1951), the applicant concluded that no evidence exists for the three faults Vernon (1951) postulated to occur in the site area (SER Figure 2.5.3-1). In addition, based on analysis of lineaments at the site location scale using the 1949 aerial photographs and the DEM developed from the 2007 LIDAR data, the applicant further concluded that no mapped lineaments intersect the LNP Units 1 and 2 site locations, although the sites are located between zones of northwest-trending lineaments and a zone of northeast-trending lineaments lies between Units 1 and 2. The applicant interpreted these northwest and northeast-trending lineaments to be due to differential carbonate dissolution localized along joints rather than along faults, and recognized that lineaments control sinkhole alignment and stream drainages in the site area.

2.5.3.2.3 Correlation of Earthquakes with Capable Tectonic Sources

FSAR Section 2.5.3.3 discusses correlation of earthquakes with capable tectonic sources within the LNP site vicinity. The applicant cross-referenced FSAR Section 2.5.2.1 for earthquake catalog data, and stated that no recorded earthquakes greater than m_b = 3.0 exist within the LNP site vicinity. The applicant concluded that no historical earthquakes or alignment of earthquakes in the site region can be associated with any mapped fault.

2.5.3.2.4 Ages of Most Recent Deformations

FSAR Section 2.5.3.4 addresses ages of most recent deformations within the LNP site vicinity and at the LNP site. The applicant stated that basement rocks, which occur about 1,330 m (4,377 ft) beneath the LNP site, record Mesozoic (251 to 65.5 Ma) deformation related to rifting associated with development of the present-day Atlantic Ocean and the Gulf of Mexico. The applicant stated that there is no well-documented evidence for faulting of the Late Cretaceous (99.6 to 65.5 Ma) or Cenozoic (6.5 Ma to present) stratigraphic sections in the site vicinity, or for the faults postulated by Vernon (1951) to displace the Middle Eocene age (48.6 to 40.4 Ma) Avon Park Formation in the site area. The applicant indicated that there is no geomorphic evidence to suggest tectonic deformation due to faulting of the bedrock surface (i.e., a marine planation surface interpreted to be older than 340,000 years) underlying Quaternary (2.6 Ma to present) terrace deposits at the site location, and that no pronounced lineaments cut across the site location to suggest a through-going fault or major fracture system.

2.5.3.2.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Structures

FSAR Section 2.5.3.5 discusses the relationship of tectonic structures in the site area to regional tectonic structures. The applicant stated that no documented faults occur within the site vicinity, but that the faults and fractures proposed by Vernon (1951) are sub-parallel to regional fracture trends observed throughout Florida. The applicant concluded that trends of fractures inferred from topographic lineaments and alignment of wetlands are consistent with trends of fractures inferred from analysis of regional lineaments and fracture sets observed in the site excavation for the CR3 site (Florida Power Corporation, 1976).

2.5.3.2.6 Characterization of Capable Tectonic Sources

FSAR Section 2.5.3.6 discusses characterization of capable tectonic sources within the LNP site vicinity. Based on review of published geologic data, interviews with technical experts knowledgeable about the site region and vicinity, and investigations performed by the applicant for the LNP COL application, the applicant concluded that no capable tectonic sources exist within the site vicinity. The applicant included the faults postulated by Vernon (1951) in the assessment of potential capable tectonic sources, concluding that no evidence exists for Quaternary deformation associated with these proposed structures.

2.5.3.2.7 Designation of Zones of Quaternary Deformation in the Site Region

FSAR Section 2.5.3.7 addresses zones of Quaternary (2.6 Ma to present) deformation in the site region. Based on review of available data and investigations preformed for the LNP COL application, the applicant concluded that no evidence exists for Quaternary tectonic deformation within the site region and site area or at the site location.

2.5.3.2.8 Potential for Surface Deformation at the Site

FSAR Section 2.5.3.8 discusses the potential for surface tectonic and non-tectonic deformation at the site. Based on review of available data and investigations preformed for the LNP COL application, the applicant stated that no capable tectonic faults or geomorphic features indicative of Quaternary (2.6 Ma to present) surface tectonic deformation occur within the site area. Consequently, the applicant concluded that the potential for surface tectonic deformation at the LNP site is negligible.

The applicant also concluded that the potential for non-tectonic surface deformation from any phenomenon other than karst-related subsidence or collapse is negligible at the site. To make this conclusion, the applicant assessed the potential effects of glacial rebound, intrusive and extrusive igneous activity, salt migration, growth faulting, and subsidence or collapse due to mining activity and gas extraction. The applicant discussed possible natural and human-induced controls on karst development, and stated that any potential for dissolution and formation of karst at the site will be mitigated by appropriate ground remediation and foundation design measures, including site-specific grouting. The applicant discussed the grouting program in detail in LNP COL FSAR Section 2.5.4.5.1, “Diaphragm Walls and Grouting.” The applicant summarized the available information reviewed as part of the karst development evaluation in FSAR Section 2.5.1.2.1.3, “Karst Terrain,” and presented the detailed evaluation of subsurface karst features in the vicinity of safety-related facilities at the LNP Units 1 and 2 site in FSAR Section 2.5.4.2, “Properties of Subsurface Materials.”

2.5.3.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

The applicable regulatory requirements for surface faulting are as follows:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying geologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR 100.23, as it relates to determining the potential for surface tectonic and non-tectonic deformations at and in the region surrounding the site.

In addition, the related acceptance criteria associated with the relevant requirements of the Commission regulations are given in Section 2.5.3 of NUREG-0800 as follows:

- Geologic, Seismic, and Geophysical Investigations: Requirements of 10 CFR 100.23 are met and guidance in RG 1.132, Revision 2; RG 1.198; and RG 1.208 is followed for this area of review if discussions of Quaternary tectonics, structural geology, stratigraphy, geochronologic methods used for age dating, paleoseismology, and geologic history of the site vicinity, site area, and site location are complete, compare well with studies conducted by others in the same area, and are supported by detailed investigations performed by the applicant.

- Geologic Evidence, or Absence of Evidence, for Surface Tectonic Deformation: Requirements of 10 CFR 100.23 are met and guidance in RGs 1.132, Revision 2; RG 1.198; and RG 1.208 is followed for this area of review if sufficient surface and subsurface information is provided by the applicant for the site vicinity, site area, and site location to confirm presence or absence of surface tectonic deformation (i.e., faulting) and, if present, to demonstrate age of most recent fault displacement and ages of previous displacements.

- Correlation of Earthquakes with Capable Tectonic Sources: Requirements of 10 CFR 100.23 are met for this area of review if all reported historical earthquakes within the site vicinity are evaluated with respect to accuracy of hypocenter location and source of origin, and if all capable tectonic sources that could, based on fault orientation and length, extend into the site area or site location are evaluated with respect to potential for causing surface deformation.

- Ages of Most Recent Deformation: Requirements of 10 CFR 100.23 are met for this area of review if every significant surface fault and feature associated with a blind fault, any part of which lies within the site area, is investigated in sufficient detail to demonstrate, or allow relatively accurate estimates of, age of most recent fault displacement and enable identification of geologic evidence for previous displacements (if such evidence exists).

- Relationship of Tectonic Structures in the Site Area to Regional Tectonic Structures: Requirements of 10 CFR 100.23 are satisfied for this area of review by discussion of structural and genetic relationships between site area faulting or other tectonic deformation and the regional tectonic framework.

- Characterization of Capable Tectonic Sources: Requirements of 10 CFR 100.23 are met for this area of review when it has been demonstrated that investigative techniques employed by the applicant are sufficiently sensitive to identify all potential capable tectonic sources, such as faults or structures associated with blind faults, within the site area; and when fault geometry, length, sense of movement, amount of total displacement and displacement per faulting event, age of latest and any previous displacements, recurrence rate, and limits of the fault zone are provided for each capable tectonic source.

- Designation of Zones of Quaternary Deformation in the Site Region: Requirements of 10 CFR 100.23 regarding designation of zones of Quaternary (< 2.6 Ma) deformation in the site region are met if the zone (or zones) designated by the applicant as requiring detailed faulting investigations is of sufficient length and width to include all Quaternary deformation features potentially significant to the site as described in RG 1.208.

- Potential for Surface Tectonic Deformation at the Site Location: To meet requirements of 10 CFR 100.23 for this area of review, information must be presented by the applicant in this section if field investigations reveal that surface or near-surface tectonic deformation along a known capable tectonic structure (i.e., a known capable tectonic feature related to a fault or blind fault) must be taken into account at the site location.

In addition, the geologic characteristics should be consistent with appropriate sections from RG 1.132, Revision 2; RG 1.198; RG 1.206; and RG 1.208.

2.5.3.4 *Technical Evaluation*

The NRC staff reviewed Section 2.5.3 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of information presented in the FSAR and the DCD completely represents the required information related to tectonic and non-tectonic surface deformation. The staff's review confirmed that information contained in the application or incorporated by reference addresses the information required for this review topic. NUREG-1793 and its supplements document the results of the staff's evaluation of the information incorporated by reference into the LNP COL application.

The staff reviewed the following information in the LNP COL FSAR:

AP1000 COL Information Item

- LNP COL 2.5-4

The NRC staff reviewed LNP COL 2.5-4 included in Section 2.5.3 of the LNP COL FSAR. LNP COL FSAR Section 2.5.3 addresses the potential for surface or near-surface tectonic and non-tectonic deformation within the site vicinity and site area and at the site location. The COL information item from AP1000 DCD, Section 2.5.3, states:

> Combined License applicants referencing the AP1000 certified design will address the following surface and subsurface geological, seismological, and geophysical information related to the potential for surface or near-surface faulting affecting the site: (1) geological, seismological, and geophysical investigations, (2) geological evidence, or absence of evidence, for surface deformation, (3) correlation of earthquakes with capable tectonic sources, (4) ages of most recent deformation, (5) relationship of tectonic structures in the site area to regional tectonic structures, (6) characterization of capable tectonic sources, (7) designation of zones of Quaternary deformation in the site region, and (8) potential for surface tectonic deformation at the site.

Based on the discussion of the potential for tectonic and non-tectonic surface deformation at the site presented in LNP COL FSAR Section 2.5.3, the staff concludes that the applicant provided the information required to satisfy LNP COL 2.5-4.

The technical information presented in FSAR Section 2.5.3 resulted from the applicant's review of existing geologic and seismicity data and published literature; discussions with individuals who have conducted recent research in and around the site area; field reconnaissance studies in the site vicinity and site area and at the site location; lineament analyses using aerial photographs and remote sensing imagery; and detailed investigations performed for the LNP site, including subsurface borings, surface geophysical testing, and downhole geophysical logging and seismic testing. The applicant also provided limited information applicable to the LNP site as derived from the FSAR prepared by Florida Power Corporation (Florida Power Corporation, 1976) for the CR3, which is located about 18 km (11 mi) southwest of the LNP COL site. Through the review of LNP COL FSAR Section 2.5.3, the staff determined whether the applicant had complied with the applicable regulations and conducted the investigations at an appropriate level of detail in accordance with RG 1.208.

NRC staff focused the review of LNP COL FSAR Section 2.5.3 on the applicant's descriptions of previous studies and data collected during those studies, as well as on the results of investigations the applicant conducted to assess the potential for surface and near-surface tectonic and non-tectonic deformation at the site. The staff visited the site in April 2009 (ML092600064), supported by technical experts from the USGS, and interacted with the applicant and its consultants in regard to the geologic, seismic, geophysical, and geotechnical investigations being conducted for the LNP COL application. During this site visit, the staff examined core samples from the initial site characterization boreholes placed at the locations of containment structures and turbine buildings for LNP Units 1 and 2, as well as exposures of the Avon Park Formation along the Waccasassa River about 25 km (16 mi) northwest of the site. Examination of the core allowed staff to assess subsurface stratigraphic relationships at the site, and the outcrops along the river permitted staff to observe and measure spacing and orientation of fractures in the Avon Park Formation. The staff also visited the site in September 2009 (ML093280825) to examine core samples from the test grouting program. The staff noted grout uptake in a single vertical fracture intersected by one of the grout boreholes. Also during the September 2009 site audit, the staff examined exposures of the Avon Park Formation at the abandoned Gulf Hammock quarry about 19 km (12 mi) north-northwest of the LNP site, which again permitted staff to observe and measure spacing and orientation of fractures in the Avon

Park Formation. In addition, in February 2010 at the applicant's records facility in Virginia, the staff examined boring logs, core photographs, and written core sample descriptions for six additional boreholes, located to be offset approximately 1.5 m (5 ft) from the position of the initial site characterization boreholes. These "offset" boreholes were drilled using controlled coring techniques to improve core recovery and further characterize soft zones postulated to mark horizons of low recovery in the initial site characterization boreholes for LNP Units 1 and 2. The two site visits and the examination of boring logs, core photographs, and core descriptions enabled the staff to assess and confirm the interpretations, assumptions, and conclusions the applicant made regarding the potential for surface and near-surface tectonic and non-tectonic deformation at the LNP site, including features related to karst development.

The following SER Sections 2.5.3.4.1 through 2.5.3.5.8 present the staff's evaluation of the information the applicant provided in LNP COL FSAR Section 2.5.3 and in responses to RAIs on that FSAR section. In addition to the RAIs addressing specific technical issues related to tectonic and non-tectonic surface deformation at the site, discussed in detail below, the staff also prepared editorial RAIs to further clarify certain descriptive statements the applicant made in the FSAR and to qualify geologic features illustrated in FSAR figures. These editorial RAIs are not discussed in this detailed technical evaluation. Also, RAIs related to geologic issues resolved in FSARs previously prepared for other sites in the CEUS are not discussed in detail in this technical evaluation for the LNP site, but rather addressed by a cross-reference to and a summary of the pertinent information used to satisfactorily resolve the issues as presented in those FSARs.

2.5.3.4.1 Geologic, Seismic, and Geophysical Investigations

LNP COL FSAR Section 2.5.3.1 summarizes the geologic, seismic, and geophysical investigations the applicant performed to assess the potential for tectonic surface deformation due to faulting within 8 km (5 mi) and 40 km (25 mi) of the site (i.e., the site area and site vicinity, respectively), as well as the potential for surface fault rupture at the LNP Units 1 and 2 site. Based on the results of these investigations, the applicant concluded that no documented tectonic faults of Quaternary age (2.6 Ma to present) occur within the site region, site vicinity, or site area, and no evidence exists for any capable (i.e., Quaternary) surface faults at the site location.

The staff focused the review of FSAR Section 2.5.3.1 on documentation of the sources used by the applicant to conclude that no capable tectonic sources occur within the site area and site vicinity, and that no evidence exists for surface faulting at the site location. In RAI 2.5.3-1, the staff asked the applicant: (a) to identify the research workers contacted; and (b) to summarize the information they provided supporting the conclusions that no capable tectonic sources occur within the site vicinity and site area and no evidence for surface faulting exists at the site location. In the response to RAI 2.5.3-1, the applicant supplied names and affiliations of the research workers who were contacted and summarized the information used to support the conclusions that no capable tectonic sources occur within the site area and site vicinity and that no evidence exists for surface faulting at the site location. The applicant emphasized the following key and current interpretations by geologists at the FGS, which strongly support these two conclusions:

- The Ocala Platform, (also referred to by some researchers as the Ocala Arch), which occurs about 14 km (8.5 mi) east of the LNP site as shown in SER Figure 2.5.3-1, is the result of sedimentary downwarping and not faulting.

- The faults postulated by Vernon (1951) to occur in the site area (i.e., unnamed Faults "A" and "B" and the Inverness fault as shown in SER Figure 2.5.3-1) based on his lineament analysis are not confirmed by more recent field data. The lineaments he associated with faulting are interpreted to be due to localized dissolution of carbonate rocks along joints.

- No known surface faults occur in the site area and none are indicated in the subsurface based on well logs, which penetrate the Avon Park Formation, the proposed foundation unit at the LNP site.

Based on the review of the applicant's response to RAI 2.5.3-1, in particular the current key interpretations provided by FGS geologists as summarized above, the staff concludes that the applicant documented the research workers contacted and summarized the pertinent information those workers provided to support the statements that no capable tectonic features occur within the site area and site vicinity and that no evidence exists for surface faulting at the site location. The staff makes this conclusion because the FGS geologists the applicant contacted are highly knowledgeable in regard to the geology and tectonic setting of Florida, and their interpretations are based on the most current data available. Furthermore, based on independent review of the technical publications provided by the applicant related to the geology and tectonic setting of Florida that support the statements made by the applicant, as well as the response to RAI 2.5.3-1, the staff further concludes that there is no reported evidence from current geologic, seismic, and geophysical investigations to indicate that capable tectonic features occur within the site area and site vicinity or that surface faulting exists at the site location. Consequently, the staff considers RAI 2.5.3-1 to be resolved.

Based on the review of LNP COL FSAR Section 2.5.3.1 and the applicant's response to RAI 2.5.3-1, the staff finds that the applicant provided a thorough and accurate description of geologic, seismic, and geophysical investigations performed to assess the potential for tectonic surface deformation due to faulting within the site area and site vicinity, as well as the potential for surface fault rupture at the LNP site, in support of the LNP COL application.

2.5.3.4.2 Geologic Evidence, or Absence of Evidence, for Surface Deformation

FSAR Section 2.5.3.2 summarizes the information the applicant presented related to the geologic evidence, or absence of evidence, for surface deformation at the site. In regard to three faults postulated by Vernon (1951) to occur in the site area (i.e., Unnamed Faults "A" and "B" and the Inverness fault, located in SER Figure 2.5.3-1), the applicant documented that no studies performed more recently than those of Vernon (1951) provide any evidence for these three faults. In addition, based on information provided by FGS geologists, the applicant indicated that the features Vernon (1951) interpreted to show evidence of surface faulting outside the site area (i.e., slickensides, which are lineations indicating direction of slip along a failure surface, and tilted bedding) are most likely the result of non-tectonic surface deformation related to karst-induced collapse. The applicant concluded that no evidence exists to suggest that these postulated features are faults, or that the features exhibit any Quaternary (2.6 Ma to present) deformation. Based on the information derived from the lineament analyses discussed in FSAR Section 2.5.3.2.1, the applicant concluded further that linear features mapped at the site location are due to localized dissolution of carbonate rocks along joints, rather than surface faulting, and that no evidence exists for tectonic surface deformation at the site. The staff focused the review of FSAR Section 2.5.3.2 on the slickensides and tilted bedding ascribed by Vernon (1951) to surface faulting; the mechanism for propagating lineaments upward through unconsolidated sediments; subsurface cross section data that may show one of the faults Vernon (1951) postulated to occur in the site area; and an inferred tectonic basin located within the site region based on FSAR Figure 2.5.3-202, but which the applicant did not discuss in the FSAR.

2.5.3.4.2.1 Slickensides and Tilted Bedding

In RAI 2.5.3-2, the staff asked the applicant to summarize the logic for stating that slickensides and tilted bedding resulted from dissolution collapse to ensure that these features do not indicate the presence of capable tectonic structures in the site area. In response to RAI 2.5.3-2, the applicant documented that FGS geologists who have extensive experience in mapping karst features interpret the slickensides and tilted bedding observed by Vernon (1951) as non-tectonic features related to karst development. Based on information provided by those geologists, the applicant indicated that the slickensides were observed to have a limited lateral extent, to be clearly associated with dissolution collapse sinkholes, and to exhibit random orientations. Therefore, the applicant concluded that these features are non-tectonic in origin and specifically related to karst development rather than faulting.

Based on review of the applicant's response to RAI 2.5.3-2, the original discussion by Vernon (1951), and the field data disclosed by FGS geologists documenting that the slickensides have a limited lateral extent, are clearly associated with dissolution collapse sinkholes, and exhibit random orientations, the staff concludes that the slickensides, and by association the tilted bedding, ascribed by Vernon (1951) to faulting are non-tectonic features related to karst development. The staff makes this conclusion because the preponderance of field evidence strongly supports a non-tectonic origin for these features. Consequently, the staff considers RAI 2.5.3-2 to be resolved.

2.5.3.4.2.2 Lineament Propagation

In RAI 2.5.3-3, the staff asked the applicant to discuss the possible non-tectonic mechanisms for propagating a lineament upward through unconsolidated sediments. This information is important to ensure that lineaments occurring in unconsolidated sediments in the site area are not related to active faulting. In the response to RAI 2.5.3-3, based on Upchurch (2008), the applicant identified the following non-tectonic mechanisms, which can cause upward propagation of fractures in competent bedrock through overlying unconsolidated sediments, without requiring the presence of faulting, which produces lineaments visible at the ground surface. The applicant incorporated changes in FSAR Sections 2.5.3.2.1 and 2.5.3.2.1.1 to include a discussion of these and other non-tectonic mechanisms for propagation of bedrock fractures upward through overlying sediments.

- Settlement of unconsolidated sediments into solution-enlarged fractures in underlying consolidated strata.

- Differential weathering or erosion caused by groundwater movement across karst surfaces.

- Differential consolidation of sediments into relict erosional features preserved in underlying unconformity surfaces.

- Growth of vegetation in clay-rich or silt-rich, moisture-holding soils located over deeper bedrock features associated with fractures.

Based on the review of the applicant's response to RAI 2.5.3-3 and the associated changes in LNP COL FSAR Sections 2.5.3.2.1 and 2.5.3.2.1.1, as well as an independent examination of the information from Upchurch (2008), the staff concludes that the applicant documented potential non-tectonic mechanisms for propagating fractures upward through unconsolidated sediments, resulting in lineaments at the ground surface that are not associated with faulting. The staff draws this conclusion based on the independent review of the information from Upchurch (2008), who is a highly credible expert on fractures, photolineaments, and mechanisms for upward propagation of fractures in bedrock through overlying unconsolidated sediments. Consequently, the staff considers RAI 2.5.3-3 to be resolved.

2.5.3.4.2.3 Postulated Subsurface Tectonic Structures

In RAI 2.5.3-4, the staff asked the applicant to discuss the cross-section data from Arthur and others (2001), illustrated in FSAR Figure 2.5.1-245, in regard to whether subsurface faulting related to a fault postulated by Vernon (1951) could be responsible for the missing subsurface limestone unit in that section. This information is important for determining if subsurface evidence exists to suggest the presence of any of the faults Vernon (1951) postulated to occur in the site area. In response to RAI 2.5.3-4, the applicant stated that Arthur and others (2001) did not interpret or discuss faulting in relation to the cross-section shown in FSAR Figure 2.5.1-245. In addition, the applicant noted that Arthur and others (2008) did not identify any faults in the LNP site area, which offset the top of the Avon Park Formation based on the isopach and structural contour maps they constructed. Given the erosional and karstic nature of the top of the Avon Park Formation, which creates a very irregular surface, the applicant concluded that there is little stratigraphic control for defining subsurface faults in the site area and that no information provided by Arthur and others (2001 and 2008) suggests the presence of faults such as those postulated by Vernon (1951).

Based on the review of the applicant's response to RAI 2.5.3-4, examination of the cross-section shown in FSAR Figure 2.5.1-245, and independent appraisal of the isopach and structural contour maps prepared by Arthur and others (2008), the staff concludes that no information provided by Arthur and others (2001 and 2008) suggests the presence of subsurface faulting in the site area, which is young enough to offset the Middle Eocene (48.6 to 40.7 Ma) age Avon Park Formation. The staff makes this conclusion because none of the data presented by Arthur and others (2001 and 2008) indicate the existence of subsurface faults, such as those postulated by Vernon (1951), in the site area. Consequently, the staff considers RAI 2.5.3-4 to be resolved.

In RAI 2.5.3-9, the staff asked the applicant to describe an inferred basin-bounding fault labeled as “D/U” in FSAR Figure 2.5.3-202, which was not discussed in the FSAR, although it occurs within the site region. This information is important to determine whether this inferred feature may be a capable tectonic structure. In response to RAI 2.5.3-9, the applicant indicated that the inferred northeast-trending fault, labeled as “D/U” is based on subsurface data, which are not definitive. Applin, who initially proposed the structure, stated that this feature occurs beneath rocks of Mesozoic (251 to 65.5 Ma) age and does not affect either Mesozoic units or younger Cenozoic (65.5 Ma to present) sediments (Applin, 1951). Based on this information from Applin (1951), the applicant concluded that this structure, if it exists, is a basement feature that does not affect rocks younger than Mesozoic. The applicant made changes to FSAR Section 2.5.1.1.4.3.1, which discuss and qualify the age of this inferred structure.

Based on the review of the applicant’s response to RAI 2.5.3-9 and the associated changes in LNP COL FSAR Section 2.5.1.1.4.3.1, the staff concludes that the inferred structure, if it exists, is a basement feature that does not affect rock units younger than Mesozoic in age. The staff draws this conclusion based on the strong field evidence cited by the applicant, which provides a Mesozoic age constraint for the feature and marks it as a structure that is older than Quaternary (2.6 Ma to present) and, therefore, not a capable tectonic structure. Consequently, the staff considers RAI 2.5.3-9 to be resolved.

Based on the review of FSAR Section 2.5.3.2, the applicant’s responses to RAIs 2.5.3-2, 2.5.3-3, 2.5.3-4, and 2.5.3-9, and the associated changes in LNP COL FSAR Sections 2.5.3.2.1, 2.5.3.2.1.1 and 2.5.1.1.4.3.1, the staff finds that the applicant provided a thorough and accurate description of the geologic evidence, or absence of evidence, for surface deformation at the site in support of the LNP COL application.

2.5.3.4.3 Correlation of Earthquakes with Capable Tectonic Sources

FSAR Section 2.5.3.3 discusses the correlation of earthquakes with capable tectonic sources within the site region and site vicinity. Based on analysis of seismic events within 320 km (200 mi) and 40 km (25 mi) of the site using an updated earthquake catalog that spanned the time frame from 1826 through December 2006, the applicant concluded that no historically-reported earthquakes or earthquake alignments can be associated with any mapped fault in the site region or site vicinity.

The staff focused the review of FSAR Section 2.5.3.3 on completeness of the seismic and tectonic information used to assess the correlation of earthquakes with capable tectonic structures in the site vicinity and site region. Based on an independent review of the updated earthquake catalog data used by the applicant, including the discussion of these data presented in FSAR Section 2.5.2.1.1, and tectonic maps showing the locations of known faults and shear zones in the site region and site vicinity, the staff concludes that no evidence exists for any correlation between earthquakes and capable tectonic structures in the site region or site vicinity.

Based on the review of FSAR Section 2.5.3.3 and the discussion in FSAR Section 2.5.2.1.1 regarding the updated earthquake data, the staff finds that the applicant provided a thorough and accurate description of the correlation of earthquakes with capable tectonic sources in support of the LNP COL application.

2.5.3.4.4 Ages of Most Recent Deformations

FSAR Section 2.5.3.4 discusses data related to the ages of most recent deformation in the site vicinity and at the LNP site. The applicant stated that there is no documented evidence for faulting of Late Cretaceous (99.6 to 65.5 Ma) or Cenozoic (65.5 Ma to present) rocks in the site vicinity, or for the faults postulated by Vernon (1951) to displace the Middle Eocene (48.6 to 40.4 Ma) Avon Park Formation in the site area. The applicant did not present information to constrain the age of the faults postulated by Vernon (1951). The applicant also stated that there is no geomorphic evidence to suggest faulting of bedrock underlying Quaternary (2.6 Ma to present) terrace deposits at the site location, and that no pronounced lineaments indicate a through-going fault or major fracture system crosscutting the site location. However, FSAR Figures 2.5.3-216, 2.5.3-218, and 2.5.3-220 show lineaments within the LNP site location based on 2007 LIDAR data, 1949 aerial photographs, and 2007 aerial photographs, respectively.

The staff focused the review of FSAR Section 2.5.3.4 on age of the faults postulated by Vernon (1951), and whether lineaments occurring at the site location may be segments of regional fracture patterns that represent geologic structures that control dissolution. In RAI 2.5.3-5, the staff asked the applicant to summarize existing information, which constrains the age of the faults postulated by Vernon (1951), particularly in regard to data indicating they are older than Quaternary, if they exist. In response to RAI 2.5.3-5, the applicant stated that the recognized experts on deformation history of the site region at the FGS do not believe the faults postulated by Vernon (1951) exist based on current field data. The applicant indicated that Arthur and others (2008) used the most current data from surface geologic mapping and water and petroleum wells to develop structure contour maps that show no faults cutting the Avon Park Formation or the overlying Ocala Limestone. The applicant reported that lineaments identified in remote sensing imagery at both a regional and site-specific scale correlate with fracture trends observed in bedrock within the site vicinity, rather than with faults. Therefore, the applicant concluded that there is no evidence to support the existence of the faults postulated by Vernon (1951) in the LNP site vicinity or site area, or that the postulated faults, if they exist, are associated with Quaternary (2.6 Ma to present) tectonic deformation. The applicant noted that this conclusion rendered it unnecessary to summarize information constraining the age of the faults postulated by Vernon (1951). The applicant provided changes in FSAR Section 2.5.3.2 to document that recent data do not support the existence of the faults postulated by Vernon (1951) to occur in the site vicinity and site area.

Based on the review of the applicant's response to RAI 2.5.3-5, which includes data provided by FGS geologists that post-date the work of Vernon (1951) and that the staff independently reviewed, as well as the associated changes in FSAR Section 2.5.3.2, the staff concludes that the more recent data do not support the existence of the faults postulated by Vernon (1951) to occur in the site vicinity and site area. The staff also concludes that no evidence exists to indicate that the lineaments identified by Vernon (1951) are indicative of Quaternary tectonic deformation. The staff makes these two conclusions because the recent field evidence provided to the applicant by FGS geologists, including the structure contour maps of Arthur and others (2008) that show no faults cutting the Avon Park Formation or the overlying Ocala Limestone as Vernon (1951) had suggested, strongly supports the interpretations that the faults postulated by Vernon (1951) do not exist and that the identified lineaments do not indicate Quaternary tectonic deformation. Consequently, the staff considers RAI 2.5.3-5 to be resolved.

In RAI 2.5.3-6, the staff asked the applicant: (a) to assess whether regional fractures may cross-cut the site location, even if discontinuously, as possibly suggested by lineaments shown in FSAR Figures 2.5.3-216, 2.5.3-218, and 2.5.3-220; and (b) whether these linear features represent geologic structures that exercise control on dissolution and sinkhole development. This information is important because fractures are known to exercise strong control on dissolution pathways in carbonate rocks. In response to RAI 2.5.3-6, the applicant cross-referenced FSAR Section 2.5.3.2.1.3 and reiterated that lineaments mapped at the site location likely reflect structurally controlled joints that have been enhanced by dissolution of carbonate and erosion. The applicant stated that the prominent northwest-trending alignment of shallow depressions located approximately 300 m (1,000 ft) west of the LNP Units 1 and 2 footprints in FSAR Figures 2.5.3-216, 2.5.3-218, and 2.5.3-220 is consistent with the strike direction of the predominant regional fracture set mapped by Vernon (1951), and with one of the predominant orthogonal outcrop-scale fracture sets mapped in exposures of the Avon Park Formation at the Gulf Hammock quarry and along the Wacasassa River, located 19 km (12 mi) and 25 km (16 mi) northwest of the site, respectively. The staff examined and measured fractures at the quarry and along the river during site visits in April and September 2009 (ML092600064 and ML093280825), and documented that outcrop-scale fractures do reflect regional fracture trends. The applicant concluded that the discontinuous character of the lineaments, the low relief exhibited by the marine terrace surface, and the absence of faulting in boreholes at the site location indicate there is no evidence to suggest that capable tectonic surface faults occur at the site. The applicant incorporated changes in FSAR Sections 2.5.3.2.1.3 and 2.5.3.4 to clarify that predominant trends of fracture sets at the site, as inferred from mapped lineaments, are consistent with regional fracture trends, stream drainage patterns, and sinkhole alignments.

Based on review of the applicant's response to RAI 2.5.3-6 and the associated changes in FSAR Sections 2.5.3.2.1.3 and 2.5.3.4, and the field observations made by staff during the April and September 2009 visits to the LNP site in regard to fracture patterns in the site vicinity, the staff concludes that lineaments mapped at the site location likely reflect structurally-controlled joints enhanced by dissolution and erosion, and that the prominent northwest-trending alignment of shallow depressions located approximately 300 m (1,000 ft) west of the LNP Units 1 and 2 footprints is consistent with the strike direction of the predominant regional fracture set mapped by Vernon (1951) and with one of the predominant outcrop-scale fracture sets. The staff makes this conclusion based on field observations made during the April and September 2009 site visits (ML092600064 and ML093280825), as well as an independent review of pertinent references the applicant cited which document the relationships between fractures and lineaments stated above. The staff also concludes that no capable tectonic surface faults occur at the site because of the field evidence cited by the applicant and directly observed by the staff related to the discontinuous expression of lineaments, the low relief of the marine terrace surface, and the absence of faulting in boreholes at the site location. Consequently, the staff considers RAI 2.5.3-6 resolved.

Based on review of FSAR Section 2.5.3.4, the applicant's responses to RAIs 2.5.3-5 and 2.5.3-6 and associated changes in LNP COL FSAR Sections 2.5.3.2 and 2.5.3.4, coupled with the observations made by staff during the April and September 2009 site visits (ML092600064 and ML093280825) in regard to regional and local-scale fracture patterns, which exist in the site vicinity, the staff finds that the applicant provided a thorough and accurate description of the ages of most recent deformation in support of the LNP COL application.

2.5.3.4.5 Relationship of Tectonic Structures in the Site Area to Regional Tectonic Structures

FSAR Section 2.5.3.5 discusses the relationship of tectonic structures in the site area to regional tectonic structures. The applicant stated that no documented bedrock faults occur within the site vicinity or site area, and that fracture trends inferred from topographic lineaments and alignment of shallow depressions and wetlands at the site location are consistent with trends of regional fractures inferred from lineament analyses.

The staff focused the review of FSAR Section 2.5.3.5 on completeness of the information used by the applicant to assess the relationship between tectonic features in the site area and regional tectonic structures. Based on the detailed up-to-date information presented in various parts of FSAR Section 2.5.3, which documents a lack of geologic evidence for tectonic faulting in the site area, as well as an independent review of that information and other published literature cited by the applicant, the staff concludes that small-scale topographic lineaments and alignment of shallow depressions and wetlands at the site location reflect the trends of regional fractures inferred from lineament analyses, rather than regional tectonic faults. The staff draws this conclusion because a preponderance of published data supports the interpretation that topographic lineaments and aligned shallow depressions and wetlands in the site area are not related to regional faults, but rather to regional fractures.

Based on the review of FSAR Section 2.5.3.5 and other FSAR sections, which document the lack of evidence for tectonic faulting in the site vicinity and site area, the staff finds that the applicant provided a thorough and accurate description of the relationship of tectonic structures in the site area to regional tectonic structures in support of the LNP COL application.

2.5.3.4.6 Characterization of Capable Tectonic Sources

FSAR Section 2.5.3.6 addresses the characterization of capable tectonic sources within the site vicinity. The applicant specifically addressed the faults postulated by Vernon (1951) to occur in the site vicinity, and documented that available data do not support the existence of these faults and that there is no evidence for Quaternary (2.6 Ma to present) deformation associated with any of these postulated structures. Therefore, the applicant concluded that no capable tectonic sources exist within the site vicinity requiring characterization.

NRC staff focused the review of FSAR Section 2.5.3.6 on completeness of the information used by the applicant to state that no capable tectonic sources requiring characterization exist with the site vicinity. Based on the detailed up-to-date information presented in various parts of FSAR Section 2.5.3, which documents a lack of geologic evidence for tectonic faulting or capable tectonic structures in the site vicinity, as well as an independent review of that information and other published literature cited by the applicant, the staff concludes that no capable tectonic sources requiring characterization exist within the site vicinity. The staff draws this conclusion because a preponderance of published data strongly supports the interpretation that no capable tectonic sources exist within the site vicinity.

Based on the review of FSAR Section 2.5.3.6 and other FSAR sections, which document the lack of evidence for capable tectonic sources at the site, the staff finds that the applicant provided a thorough and accurate description regarding characterization of capable tectonic sources within the site vicinity in support of the LNP COL application.

2.5.3.4.7 Designation of Zones of Quaternary Deformation in the Site Region

FSAR Section 2.5.3.7 addresses the designation of zones of Quaternary (2.6 Ma to present) deformation in the site region, which may require detailed investigations. The applicant cross-referenced the detailed information on site geology presented in FSAR Section 2.5.1.2 and concluded that, based on both surface and subsurface data, no zones of Quaternary deformation requiring further investigation occur within the site region, site area, or at the LNP site location. However, the applicant did not summarize the pertinent results from the subsurface investigations, which supported this conclusion.

NRC staff focused the review of FSAR Section 2.5.3.7 on documentation of subsurface data sources used by the applicant to support the conclusion that no zones of Quaternary deformation requiring further investigation occur within the site region, site area, or at the LNP site location. In RAI 2.5.3-7, the staff asked the applicant to summarize the data derived from subsurface investigations that support this conclusion. In the response to RAI 2.5.3-7, the applicant stated that FSAR Section 2.5.4.2 presents the results of the extensive geotechnical boring program conducted at the LNP site to investigate subsurface rock conditions, and that no faults or other tectonic structures were revealed in geologic logs for more than 100 borings. The applicant cross-referenced the response to RAI 2.5.1-10, which documented that there was no evidence for faults or associated tectonic structures in televiewer logs from eight geotechnical borings drilled to a maximum depth of 152 m (500 ft) below the ground surface within the nuclear island footprint. The applicant also referred to FSAR Section 2.5.1.2.5.2.1, which describes subsurface organic-rich marker beds in the Avon Park Formation at the LNP site, detected in geophysical logs and core samples from four boreholes, and stated that these beds do not display abrupt vertical offsets as would be expected if significant tectonic deformation had occurred. The applicant provided changes in FSAR Section 2.5.3 to include the additional information about site-specific subsurface observations discussed in the response to RAI 2.5.3-7.

During the site visits in April and September 2009 (ML092600064 and ML093280825), staff examined core samples from geotechnical boreholes drilled at the LNP site and noted that no evidence existed for subsurface faults at the site. In addition, in February 2010, the staff examined boring logs, core photographs, and written core sample descriptions for six additional boreholes located to further characterize zones of low recovery observed in boreholes drilled during the initial site characterization phase for LNP Units 1 and 2. Examination of these core logs, photographs, and descriptions also did not reveal the presence of subsurface faults at the site. Therefore, based on the review of the applicant's response to RAI 2.5.3-7, including the revisions in FSAR Section 2.5.3, as well as the direct observations made by staff during the April and September 2009 site visits and the results of the examination of core logs, core photographs, and core sample descriptions in February 2010, the staff concludes that the applicant properly summarized the subsurface information used to determine that no zones of Quaternary deformation, which would require further investigation occur within the site region, site area, or at the LNP site location.

Based on the review of FSAR Section 2.5.3.7, the applicant's response to RAI 2.5.3-7 and the associated changes in FSAR Section 2.5.3, as well as the direct examination by staff of core samples from the LNP site during the April and September 2009 site visits and of core logs, core photographs, and core sample descriptions in February 2010, the staff finds that the applicant provided a thorough and accurate description in regard the designation of Quaternary deformation zones in the site region in support of the LNP COL application.

2.5.3.4.8 Potential for Surface Deformation at the Site

LNP COL FSAR Section 2.5.3.8 discusses the potential for surface tectonic deformation, as well as non-tectonic surface deformation related to karst development and phenomena other than karst-induced collapse or subsidence, at the site. In FSAR Section 2.5.3.8.1, the applicant concluded that the potential for surface tectonic deformation at the site is negligible because no capable tectonic structures or geomorphic features indicative of Quaternary (2.6 Ma to present) deformation exist within the LNP site area. Also in FSAR Section 2.5.3.8.1, the applicant indicated that excavations for all safety-related structures for LNP Units 1 and 2 would be mapped in detail, and the NRC notified immediately if previously unrecognized geologic features that may represent a hazard to the facilities were identified. In addition, the applicant stated that any deformation features observed in the excavations would be characterized to assess the potential for surface deformation and ground motion following guidance in RG 1.208. These actions are identified as License Condition 2-1 under SER Section 2.5.3.5. In FSAR Section 2.5.3.8.2.1, the applicant stated that the potential for non-tectonic surface deformation at the site is negligible, except for phenomena related to karst-induced collapse or subsidence. In FSAR Section 2.5.3.8.2.2, the applicant specifically addressed the potential for karst-related non-tectonic surface deformation and concluded that karst-induced collapse and subsidence pose a potential geologic hazard at the LNP site.

NRC staff focused the review of FSAR Section 2.5.3.8 on completeness of the information the applicant used to assess the potential for surface tectonic and non-tectonic deformation at the site. In regard to tectonic surface deformation, based on the detailed up-to-date information presented in various parts of FSAR Section 2.5.3, which documents a lack of geologic evidence for surface or subsurface tectonic faulting in the site area, as well as an independent review of that information and other published literature cited by the applicant, the staff concludes that the potential for surface tectonic deformation at the site is negligible. For non-tectonic surface deformation, based on detailed up-to-date information presented in various parts of FSAR Section 2.5.3, which documents a lack of geologic evidence for non-tectonic surface deformation except for phenomena associated with karst-related collapse and subsidence, as well as an independent review of that information and other published literature cited by the applicant, the staff concludes that the potential for non-tectonic surface deformation exists only in connection with karst-related collapse and subsidence. The staff draws these two conclusions because a preponderance of published data strongly supports the interpretations that the potential for surface tectonic deformation at the site is negligible, and that phenomena associated with karst-related collapse and subsidence provide the only potential for non-tectonic surface deformation. In addition, detailed examination by staff during the April and September 2009 site visits of core samples taken from boreholes at the LNP site revealed only a few fractures and no extensive dissolution features or faults in support of the applicant's conclusions regarding tectonic and non-tectonic surface deformation. Furthermore, in February 2010, the staff examined boring logs, core photographs, and written core sample descriptions for six additional boreholes located to further characterize postulated soft zones, which were noted in boreholes drilled during the initial site characterization studies for LNP Units 1 and 2. Examination of these core logs, photographs, and descriptions likewise did not reveal the presence of either subsurface faulting or extensive dissolution cavities at the site.

Based on the review of FSAR Section 2.5.3.8 and other FSAR sections, which document the lack of evidence for surface tectonic faulting and the possibility of non-tectonic surface deformation related to karst development at the site, as well as the examination by staff during the April and September 2009 site visits of core samples from the LNP site and examination of core logs, photographs, and descriptions in February 2010, the staff finds that the applicant provided a thorough and accurate description of the potential for tectonic and non-tectonic surface deformation at the site in support of the LNP COL application.

2.5.3.5 *Post Combined License Activities*

Staff identified the following License Condition as the responsibility of the COL licensee in SER Section 2.5.3.4.8 ("Potential for Surface Deformation at the Site"). This License Condition relates to geologic mapping of both tectonic and non-tectonic (i.e., karst-induced collapse and subsidence) surface deformation features at the site.

- License Condition (2-1) – The licensee shall perform detailed geologic mapping of the excavations for LNP Units 1 and 2 nuclear island structures; examine and evaluate geologic features discovered in excavations for safety-related structures other than those for the Units 1 and 2 nuclear islands; and notify the Director of the Office of New Reactors, or the Director's designee, once excavations for LNP Units 1 and 2 safety-related structures are open for examination by NRC staff.

2.5.3.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The staff confirmed that the applicant addressed the required information related to tectonic and non-tectonic surface deformation, and that there is no outstanding information expected to be addressed in the LNP COL FSAR related to this topic. The results of the staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the staff has reviewed the information in LNP COL 2.5-4 and finds that the applicant provided a thorough characterization of the potential for tectonic and non-tectonic surface and near-surface deformation, including faulting, at the LNP site, as required by 10 CFR 100.23 and 10 CFR 52.79(a)(1)(iii). Based on the review of the geologic and seismic information gathered by the applicant during the regional and site-specific investigations and presented in LNP COL FSAR Section 2.5.3, the staff concludes that the applicant performed its investigations in accordance with 10 CFR 100.23 and 10 CFR 52.79(a)(1)(iii) and followed guidance provided in RG 1.208. The staff also concludes that the applicant established an adequate basis to state that no known capable tectonic sources exist in the site vicinity, which would cause surface or near-surface deformation in the site area, and that the potential for surface or near-surface non-tectonic deformation in the site area is negligible, with the exception of karst-induced collapse and subsidence. Therefore, the staff concludes that the proposed LNP Units 1 and 2 COL site is acceptable from the perspective of surface and near-surface tectonic deformation and meets the requirements of 10 CFR 100.23 and 10 CFR 52.79(a)(1)(iii).

2.5.4 Stability of Subsurface Materials and Foundations

2.5.4.1 *Introduction*

Section 2.5.4 of this SER presents information on the static and dynamic stability of subsurface materials and foundations for the LNP Units 1 and 2 COL site. The properties and stability of the soil and rock underlying the site are important to the safe design and siting of the plant. The information related to the stability of subsurface materials and foundations covers the following specific areas: (1) geologic features in the vicinity of the site; (2) static and dynamic engineering properties of soil and rock strata underlying the site; (3) the relationship of the foundations for safety-related facilities and the engineering properties of underlying materials; (4) results of seismic surveys, including in-hole explorations; (5) safety-related excavation and backfill plans and engineered earthwork analysis and criteria; (6) groundwater conditions and piezometric pressure in all critical strata as to affect the loading and stability of foundation materials; (7) responses of site soils or rocks to dynamic loading; (8) liquefaction potential and consequences of liquefaction of all subsurface soils, including the settlement of foundations; (9) earthquake design bases; (10) results of investigations and analyses conducted to determine foundation material stability, deformation and settlement under static conditions; (11) criteria, references, and design methods used in static and seismic analyses of foundation materials; and (12) techniques and specifications to improve subsurface conditions, which are to be used at the site to provide suitable foundation conditions, and any additional information deemed necessary in accordance with 10 CFR Part 52.

2.5.4.2 *Summary of Application*

Section 2.5 of the LNP COL FSAR, Revision 9, incorporates by reference Section 2.5.4 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Section 2.5.4, the applicant provided site-specific information to address the following:

AP1000 COL Information Items

- LNP COL 2.5-5

The applicant provided additional information in LNP COL 2.5-5 to resolve COL Information Item 2.5-5 (COL Action Item 2.5.1-1). LNP COL 2.5-5 addresses the provision of site-specific information regarding the underlying site conditions and geologic features, including site topographical features and the locations of seismic Category I structures.

- LNP COL 2.5-6

The applicant provided additional information in LNP COL 2.5-6 to resolve COL Information Item 2.5-6 (COL Action Item 2.6-3). LNP COL 2.5-6 addresses the properties of the foundation rock to be within the range considered for the design of the nuclear island basemat.

- LNP COL 2.5-7

The applicant provided additional information in LNP COL 2.5-7 to resolve COL Information Item 2.5-7 (COL Action Item 2.5.4-1). LNP COL 2.5-7 addresses the information concerning the extent (horizontal and vertical) of seismic Category I excavations, fills, and slopes.

- LNP COL 2.5-8

The applicant provided additional information in LNP COL 2.5-8 to resolve COL Information Item 2.5-8 (COL Action Item 2.4.1-1). LNP COL 2.5-8 addresses the ground water conditions relative to the foundation stability of the safety-related structures at the site.

- LNP COL 2.5-9

The applicant provided additional information in LNP COL 2.5-9 to resolve COL Information Item 2.5-9 (COL Action Item 2.5.4.3-1). LNP COL 2.5-9 addresses the provision of demonstrating that the potential for liquefaction is negligible.

- LNP COL 2.5-10

The applicant provided additional information in LNP COL 2.5-10 to resolve COL Information Item 2.5-10 (COL Action Item 2.6-4). LNP COL 2.5-10 addresses the verification that the minimum allowable bearing capacity of the site is greater than that specified in the AP1000 DCD with an adequate factor of safety.

- LNP COL 2.5-11

The applicant provided additional information in LNP COL 2.5-11 to resolve COL Information Item 2.5-11 (COL Action Item 2.5.2-2). LNP COL 2.5-11 addresses the methodology used in determination of static and dynamic lateral earth pressures and hydrostatic groundwater pressures acting on plant safety-related facilities using soil parameters as evaluated in previous sections.

- LNP COL 2.5-12

The applicant provided additional information in LNP COL 2.5-12 to resolve COL Information Item 2.5-12 (COL Action Item 2.5.5-1). LNP COL 2.5-12 addresses the rock characteristics affecting the stability of the nuclear island including foundation rebound, settlement, and differential settlement.

- LNP COL 2.5-13

The applicant provided additional information in LNP COL 2.5-13 to resolve COL Information Item 2.5-13 (COL Action Item 2.6-5). LNP COL 2.5-13 addresses the provision for instrumentation for monitoring the performance of the foundations of the nuclear island, along with the location for benchmarks and markers for monitoring the settlement.

- LNP COL 2.5-16

The applicant provided additional information in LNP COL 2.5-16 to resolve COL Information Item 2.5-16. LNP COL 2.5-16 addresses the verification that both total and differential settlements of the nuclear island, and the differential settlements between the nuclear island and other buildings do not exceed the AP1000 standard design.

- LNP COL 2.5-17

In a letter dated July 21, 2009, Westinghouse proposed COL Information Item 2.5-17 to provide a waterproofing system used for the below grade, exterior walls exposed to flood and groundwater under seismic Category I structures. COL Information Item 2.5-17 states that:

> The Combined License applicant will provide a waterproofing system used for the below grade, exterior walls exposed to flood and groundwater under seismic Category I structures. Waterproofing membrane should be placed immediately beneath the upper Mud Mat, and on top of the lower Mud Mat. The performance requirements to be met by the COL applicant for the waterproofing system are described in subsection 3.4.1.1.1.1.

Evaluation of the waterproofing capability of the system presented in LNP COL 2.5-17 occurs in Section 3.8 of this SER. The evaluation of the system's ability to meet the seismic requirements outlined in DCD Section 3.4.1.1.1.1 is located in Section 3.8 of this SER. The inspections, tests, analyses, and acceptance criteria (ITAAC) for the waterproof membrane is evaluated in Section 14.3 of this SER.

In addition, this LNP COL FSAR section addresses Interface Item 2.12, related to V_S, and Interface Item 2.13, related to the required bearing capacity of foundation materials.

In LNP COL FSAR Section 2.5.4, the applicant described the geotechnical explorations performed at the LNP site to determine the in-situ soil and rock properties and obtain samples for laboratory testing, the laboratory tests conducted to confirm the soil and rock properties, and the analyses made to determine the acceptability of the LNP Units 1 and 2 site as compared to the AP1000 DCD site requirements.

2.5.4.2.1 Geologic Features

LNP COL FSAR Section 2.5.4.1 summarizes the geologic features present at the LNP Units 1 and 2 sites, including those features that could relate to permanent ground deformations or foundation instability; areas of potential or actual subsurface subsidence, solution activity, uplift, or collapse; zones of alteration, irregular weathering, or structural weakness; unrelieved stresses in bedrock; rocks or soils that may be unstable; and the history of deposition and erosion. The applicant referred to FSAR Sections 2.5.1 and 2.5.3 for additional details of the geology and potential for surface faulting, respectively.

2.5.4.2.2 Properties of Subsurface Materials

FSAR Section 2.5.4.2 describes the subsurface investigation activities performed to characterize the soil and rock underlying the safety-related structures at the LNP site. All elevations given are with respect to the North American Vertical Datum of 1988.

2.5.4.2.2.1 Description of Investigation Activities

FSAR Section 2.5.4.2.1 describes the combination of field activities and laboratory tests performed and the engineering standards used to obtain the engineering properties of soils and rock at the LNP site.

2.5.4.2.2.2 Soil Boring and Rock Coring

The applicant described the initial, main and supplemental phases of the site investigations. During the initial phase, the applicant used sonic drilling to drill ten boreholes to characterize the subsurface conditions and conduct geophysical logging. As part of the main phase, the applicant drilled ninety boreholes to obtain soil and rock samples for laboratory testing. Based on the results of the initial and main phases, the applicant concluded that the subsurface conditions were potentially non-uniform. The applicant conducted a supplemental phase to drill eighteen boreholes to better characterize the subsurface conditions and the potential non-uniformity of the LNP site. An additional supplemental phase of drilling referred to as the "Offset Borings" (O-series) was drilled during the COL application review in response to requests for additional information. These borings are discussed in detail in Section 2.5.4.4 of this SER. The A-, B-, AD- and O-series borings were drilled within or in close proximity to the footprint of the nuclear islands and were relied on by the staff in the evaluation of the foundation conditions. SER Figure 2.5.4-1 shows, in plan, the relationship between LNP Units 1 and 2 and the boring locations.

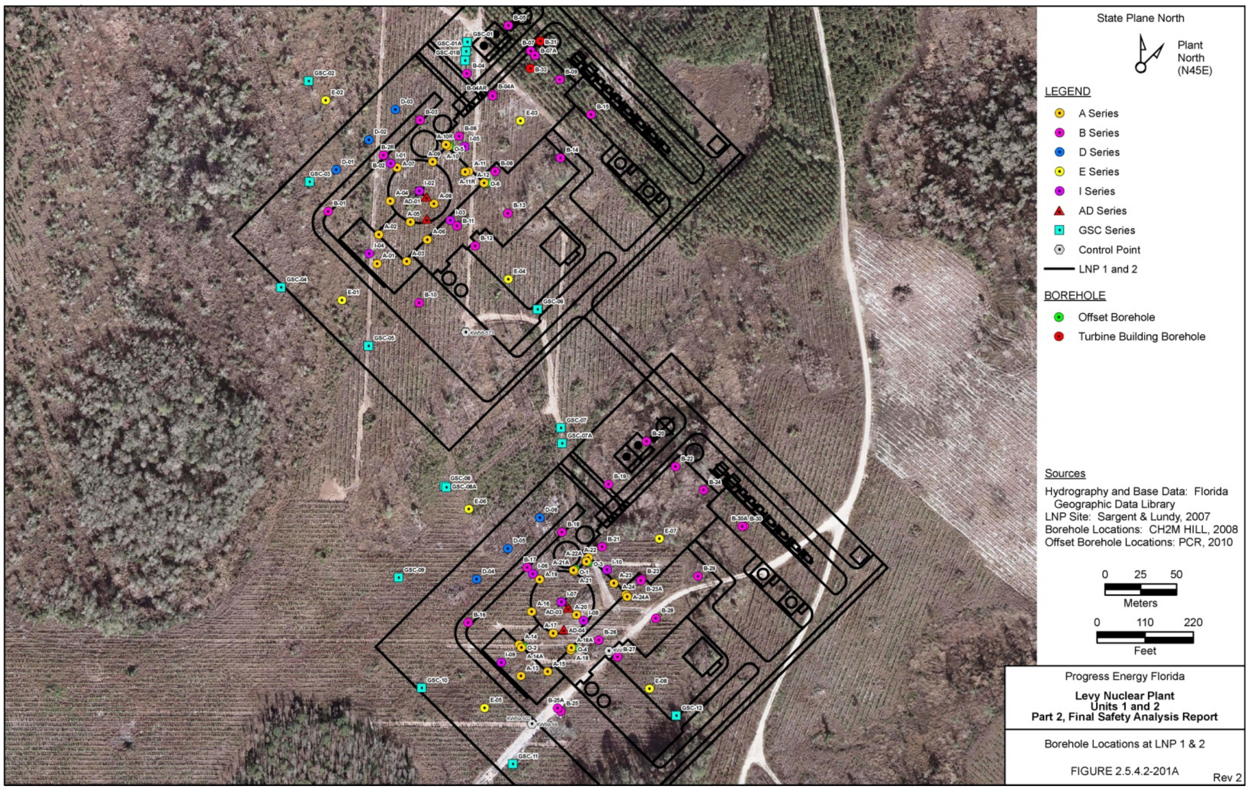


Figure 2.5.4-1. LNP 1 and 2 Boring Location Plan
(FSAR Figure 2.5.4.2-201A)

Criteria for Selection of Borehole Locations and Depths

The applicant selected borehole locations and depths for the initial and main phases following the criteria provided in RG 1.132. The applicant selected the location of supplemental phase borings based on the final orientation of the buildings and the need to obtain additional information for engineering analysis purposes. The applicant advanced borings in the supplemental phase to depths exceeding the maximum dimension of the nuclear island of 78 m (256 ft).

2.5.4.2.2.3 Drilling and Sampling Method

For the initial phase, the applicant used a Rotosonic (sonic) drilling method to continuously sample the soil and rock for visual classification purposes. During the main phase, the applicant used the mud rotary drilling method to advance the boring to collect representative disturbed soil samples using standard penetration tests (SPT) methods in accordance with American Society for Testing and Materials (ASTM) D 1586-99 and to obtain rock core samples using NQ- and HQ-sized, double tube diamond-tipped rock coring tools, in accordance with ASTM D2113. The applicant was unable to obtain undisturbed samples of soil due to the granular nature of the soil.

2.5.4.2.2.4 Field Observations, Logs, and Field Tests

The applicant conducted field investigation activities to characterize the types of soil and rock, soil consistency, rock strength and stiffness. The applicant recorded observations on boring logs, including visual descriptions of soil samples and rock cores, SPT N-values, field measurements of rock soundness and strength, rock core recovery, rock quality designations (RQDs), and R-values. Field tests such as field point load tests (PLTs) on rock cores, and in-situ rock pressuremeter tests (PMTs) in uncased boreholes are summarized in the FSAR.

2.5.4.2.2.5 Basis for Selection of Field Rock Hardness and Strength Tests

The applicant estimated the rock consistency at the LNP site using various field and laboratory tests, including unconfined compressive strength (UCS) tests, field R-scale tests, field PLTs, and downhole PMTs. The UCS tests provided the primary intact rock strength, while the R-scale tests and PLTs provided a check of the UCS data. The applicant performed PMTs in two boreholes at various depths to estimate the in-situ elastic modulus of the rock.

2.5.4.2.2.6 Geophysical Surveys

The applicant performed a series of seismic and non-seismic surveys, including suspension P-S velocity logging, downhole velocity logging, acoustic televiewer surveys, natural gamma measurements, gamma-gamma measurements, neutron-neutron measurements, and induction measurements. The applicant used the V_s profiles from the seismic surveys to determine the GMRS and estimate the stiffness of the Avon Park limestone. The non-seismic geophysical survey data was used to evaluate the stratigraphy at the site.

2.5.4.2.2.7 Management of Soil and Rock Core Samples

FSAR Section 2.5.4.2.1.4 describes the management of soil and rock samples. The applicant stored soil samples recovered by SPT sampling in watertight jars and routine-care rock core samples in core boxes kept at onsite long-term storage facilities. The applicant shipped special-care rock core samples to laboratory facilities for testing.

2.5.4.2.2.8 Laboratory Testing of Soil and Rock

In FSAR Section 2.5.4.2.1.5, the applicant described the laboratory testing of soil and rock at the LNP site, including a summary of the laboratory tests performed and the criteria for the selection of soil and rock samples.

Laboratory Tests Performed

The applicant presented the results of the laboratory tests performed on the special-care intact rock cores, which included UCS tests with axial and radial strain measurement, triaxial compression tests and split-tensile strength tests, petrographic examinations, and x-ray fluorescence examinations. The applicant also performed index tests, resistivity tests, pH tests, and organic content tests on SPT soil overburden samples. The applicant performed additional soil tests on two non-lithified and highly organic soil-like samples sandwiched within the Avon Park limestone at depths significantly below the foundations of the nuclear islands at the LNP site.

2.5.4.2.2.9 Criteria for Selection of Soil Samples for Laboratory Testing

The applicant classified any material that could be penetrated and sampled using SPT methods as “soil” or “soil-like.” The applicant plans to excavate these materials within the nuclear island footprint to the top of rock designated at an elevation (El.) of -7.3 m (-24 ft), prepare the rock surface with dental concrete, and overlay the Avon Park limestone with a 10.7 m (35 ft) thick roller compacted concrete (RCC) bridging mat. The applicant concluded that the laboratory tests on these materials are only relevant for existing soils outside the limits of the nuclear island where they are not excavated and replaced by more stable materials.

2.5.4.2.2.10 Criteria for Selection of Rock Core Samples for Laboratory Testing

The applicant collected special-care rock core samples in order to target specific elevation ranges, characterize different rock types, span the range of apparent rock core soundness, and obtain information on identified rock layers.

2.5.4.2.2.11 Results of Soil and Rock Tests Obtained from Field Investigations

The applicant recorded SPT blow counts (N) in the soil overburden and obtained disturbed samples from the split-spoon sampler for identification of soil and soil-like materials. Beginning at the top of the Avon Park limestone the applicant used a double-tube core barrel to recover rock cores. The applicant noted core recovery, RQD, R-scale values, PLT indices, time of drilling, water circulation loss, rod drops and descriptions of the recovered core on the core logs. Field PMT data in rock was obtained in two uncased boreholes during the field exploration.

2.5.4.2.2.12 Standard Penetration Test Blow Counts (N)

The applicant recorded SPT blow counts (N), the penetration resistance of the soil measured in blows per foot (bpf), at 0.76 to 1.5 m (2.5 to 5.0 ft) intervals from the existing ground surface to the depth of the top of rock in accordance with ASTM D1586 (1999). The applicant used the N-value to characterize three distinct soil layers at LNP Units 1 and 2: top layer S-1 with N-values of less than 30 bpf, intermediate soil S-2 with N-values between 30 and 50 bpf, and bottom soil S-3 with N-values greater than 50 bpf.

2.5.4.2.2.13 Rock Quality Designation, Rock Mass Quality, and Karst Features

The applicant determined the RQD, which is a rock soundness index, based on the length of recovered core pieces greater than 10 cm (4 in) compared to the total length of recovered core. The applicant used RQD values in combination with other data to delineate distinct rock layers. The applicant determined that the karst features identified in the core borings were either voids or soil-infilled based on drilling criteria, such as time of drilling, water circulation loss, and driller comments regarding rig behavior. Subsequent to the offset boring program, the applicant concluded that postulated infilled features are severely weathered or degraded dolomite with properties consistent with the Avon Park Formation.

2.5.4.2.2.14 R-Scale Strength Values

The applicant stated that the R-scale values provide a qualitative indication of rock strength and rated the rock at the site as R2 (weak rock) or stronger. FSAR Appendix 2BB reports the R-values recorded in the rock core logs.

2.5.4.2.2.15 Rock Pressuremeter Test (PMT) Modulus (Epmt)

LNP COL FSAR Section 2.5.4.2.2.5 states that the rock PMTs were performed at various depths in two boreholes at LNP Units 1 and 2. LNP COL FSAR Table 2.5.4.2-206 presents the results, which show that the Young's modulus values range from 6.9 to 1,689 megaPascal (MPa) (1 to 245 kips per square inch (ksi)) and 213 to 2,171 MPa (31 to 315 ksi) at LNP Units 1 and 2, respectively. Because the nature of the soft rock prevented the applicant from completing a sufficient number of pressure stages to provide complete and accurate results, the applicant concluded that the Young's modulus values obtained from the PMT were "worst case" estimates and, therefore, were not used in the engineering analyses.

2.5.4.2.2.16 Hydraulic Conductivity Tests

The applicant installed monitoring wells at the LNP site to monitor the seasonal fluctuation in ground water elevations and observation wells to assess the hydraulic conductivity of soil and rock. The applicant also measured the hydraulic gradients from the onsite ground water monitoring wells and referred to FSAR Section 2.4.12.2 for a more detailed description of the ground water hydrology at LNP Units 1 and 2.

2.5.4.2.2.17 Criteria for Soil Depth and Top of Rock

Because the top of rock was not distinct, the applicant defined the "top of rock" as the first occurrence of rock core and subsequent rock core runs recovering at least 50 percent of the core and having a minimum RQD of 25 percent in each core run.

2.5.4.2.2.18 Results of Soil Laboratory Tests

Based on the results of the petrographic analyses, the applicant concluded that the Avon Park limestone was dolomitized making it more resistant to future development of karst features.

2.5.4.2.2.19 Rock and Soil Properties for Use in Engineering Analyses

FSAR Section 2.5.4.2.4 summarizes rock and soil properties obtained from the field and laboratory tests. SER Table 2.5.4-1 compiles the elastic modulus, Poisson's ratio, rock mass shear strength parameters developed using the Hoek-Brown criteria, V_s and compression wave (V_p) velocity obtained from the suspension P-S velocity logging and the rock mass modulus derived from three data sources. The applicant noted that the rock mass elastic modulus (E_{rm}) values based on UCS data were 40 to 90 percent lower than those estimated using the small strain seismic data. SER Table 2.5.4-2 presents the soil properties and strength parameters derived from empirical relationships.

Table 2.5.4-1. Summary of Rock Samples
(Data Compiled from FSAR Tables 2.5.4.2-211 through 2.5.4.2-215)

	LNP 1			LNP 2			
	SAV*-1	SAV-2	SAV-3	NAV**-1	NAV-2	NAV-3	NAV-4
UCS, Elastic Moduli, Poisson's Ratio and Index Test Results							
Average UCS, MPa (psi)	25.9 (3,760)	5.07 (736)	25.4 (3,690)	16.6 (2,414)	20.2 (2,938)	16.9 (711)	17.4 (2,526)
Poisson's Ratio – Secant	0.29	0.50	0.22	0.34	0.30	0.36	0.16
Bulk Density, kg/m^3 (pcf)	2,210 (138)	2,002 (125)	2,306 (144)	2,146 (134)	2,178 (136)	1,890 (118)	2,162 (135)
Moisture Content, %	10	23	13	14	11	23	20
Poisson's Ratio – Tangent	0.36	0.51	0.32	0.44	0.37	0.53	0.16
Tensile Strength Test Results							
Tensile Strength, kPa (psi)	4,840 (702)	n/a	4,536 (658)	1,640 (238)	3,874 (562)	158.5 (23)	1,130 (164)
Bulk Density, kg/m^3 (pcf)	2,290 (143)	n/a	2,418 (151)	2,098 (131)	2,194 (137)	1,954 (122)	1,938 (121)
Moisture Content, %	9	n/a	10	17	12	27	21
Hoek-Brown Rock Mass Strength Parameters							
Unit Weight, kg/m^3 (pcf)	2,210 (138)	2,002 (125)	2,306 (144)	2,146 (134)	2,178 (136)	1,890 (118)	2,162 (135)
Representative UCS of Intact Rock, MPa (psi)	25.5 (3,700)	4.82 (700)	24.8 (3,600)	16.5 (2,400)	19.9 (2,900)	4.82 (700)	17.2 (2,500)
GSI	31	21	27	37	38	22	31
Rock Mass Cohesion, kPa (psi)	186 (27)	144 (21)	565 (82)	179 (26)	365 (53)	137 (20)	496 (72)
Rock Mass Friction Angle	24	15	22	24	25	16	21

Table 2.5.4-1. Summary of Rock Samples (Data Compiled from FSAR Tables 2.5.4.2-211 through 2.5.4.2-215)

	LNP 1			LNP 2			
	SAV*-1	SAV-2	SAV-3	NAV**-1	NAV-2	NAV-3	NAV-4
Suspension Logging							
V_s, m/s (fps)	1,198 (3,932)	893 (2,932)	1,170 (3,839)	1,115 (3,660)	1,406 (4,614)	943 (3,097)	1,207 (3,963)
V_p, m/s (fps)	893 (9601)	2,366 (7,763)	2,756 (9,045)	2,549 (8,365)	3,022 (9,916)	2,440 (8,008)	2,775 (9,105)
Poisson's Ratio	0.39	0.41	0.38	0.38	0.35	0.41	0.38
Young's Modulus, kPa (ksi)	9,507 (1,379)	4,660 (676)	8,990 (1,304)	7,535 (1,093)	11,948 (1,733)	4,881 (708)	8,928 (1,295)
E_{rm} Values by Method Used, kPa (ksi)							
V_s, m/s (fps)	690 (4,757)	338 (2,330)	652 (4,495)	547 (3,771)	867 (5,977)	354 (2,440)	647 (4,460)
Rock PMT	834 (121)	-	-	427 (62)	-	-	-
UCS Testing [a]	1,048 (152)	268 (39)	1,640 (238)	875 (127)	1,758 (255)	351 (51)	2,868 (416)
UCS Testing [b]	1,172 (170)	179 (26)	1,075 (156)	979 (142)	1,489 (216)	234 (34)	1,799 (261)

* SAV is a rock unit of the Avon Park formation at the south reactor site
** NAV is a rock unit of the Avon Park formation at the north reactor site
[a] Hoek and Diederichs (2006)
[b] Yang (2006)

Table 2.5.4-2. **Estimated Properties of Soils above the Top of Rock (Modified from FSAR Tables 2.5.4.2-216 and 2.5.4.2-217)**

	North Reactor LNP 2			South Reactor LNP 1		
	S-1	S-2	S-3	S-1	S-2	S-3
Based on Laboratory Index Properties						
Avg. f'_{cv}, deg.	31	30	29	31	n/a	-
OCR	1.7	1.0	2.0	4.4	n/a	-
s_u kPa (psf)	21.4 (449)	30.4 (636)	70.4 (1,471)	36.8 (769)	n/a	-
C_c	0.31	0.34	0.38	0.30	n/a	-
C_r	0.05	0.07	0.08	0.06	n/a	-
C_{ea}	0.003	0.004	0.004	0.002	n/a	-
Based on SPT N-values						
Mean SPT N-value, bpf	10	43	85	9	43	82
N_{60}, bpf	11	45	86	11	52	86
Moist Unit Weight, kg/m^3 (pcf)	1,762 (110)	1,922 (120)	2,082 (130)	1,762 (110)	1,922 (120)	2,082 (130)
Relative Density, %	25	50	90	25	50	90
Effective Friction Angle	28	31	36	28	31	36
Effective Cohesion	0					
Elastic Modulus[a], MPa (psi)	5.57 (808)	22.8 (3,307)	43.5 (6,319)	5.57 (808)	26.3 (3,821)	43.5 (6,319)
Elastic Modulus[b], MPa (psi)	11.9 (1,736)	27.7 (4,028)	47.8 (6,944)	11.4 (1,667)	27.7 (4,028)	46.4 (6,736)
Elastic Modulus[c], MPa (psi)	4.70 (683)	14.8 (2,148)	27.1 (3,940)	4.41 (640)	14.8 (2,148)	26.2 (3,812)
Shear Modulus, MPa (psi)	2.43 (353)	9.57 (1,389)	17.2 (2,498)	2.43 (353)	11.0 (1,605)	17.2 (2,498)

[a] Kulhawy and Mayne (1990)
[b] Webb (1969)
[c] Begemann (1974)

2.5.4.2.3 Foundation Interfaces

FSAR Section 2.5.4.3 describes the site layout, plant orientation, surface conditions, and other site details. The applicant located LNP Units 1 and 2 in previously underdeveloped, vegetated areas. The soil profile overlying the Avon Park limestone formation consists of 3 distinct soil layers, S-1, S-2 and S-3. The soil layers were differentiated based on the SPT N-value results, which measure the penetration resistance of the soil over the sampling interval, typically 0.5 m (1.5 ft). Penetration resistance is an index of the compactness of the layer, when other factors such as overburden pressure hammer energy are taken into account. The results of the SPT indicate layer S-1 is very loose to loose, S-2 is intermediate in compactness to S-1 and S-3, and S-3 is dense to very dense. The lower two layers, S-2 and S-3, are thought to be more compact partially due to cementation. Competent Avon Park limestone underlies the soil layers and was identified by the refusal of the SPT to penetrate the limestone. The depth to the competent

Avon Park limestone is variable across the site as shown in the cross-sections in the LNP COL FSAR.

The applicant stated that it will raise the existing ground surface at an El. of 12.2 to 13.4 m (40 to 44 ft) to a final site grade at an El. of 15.5 m (51 ft). The applicant included provisions for drainage in the site grading plan. SER Figure 2.5.4-1 shows the boring locations within the LNP Units 1 and 2 power blocks. SER Figure 2.5.4-2 shows the geologic interpretation at the LNP Unit 1 Plant North-South cross-section with the soil layers, S-1, S-2 and S-3 overlying the Avon Park limestone layers, SAV-1, SAV-2 and SAV-3, at the south reactor site. Similar figures were provided in the LNP COL FSAR for LNP Unit 2, the north reactor site, where the Avon Park limestone was subdivided into 4 layers, NAV-1, NAV-2, NAV-3 and NAV-4. The limestone subdivision was based on the results of the geophysical testing, primarily the results of the suspension P-S velocity logging survey.

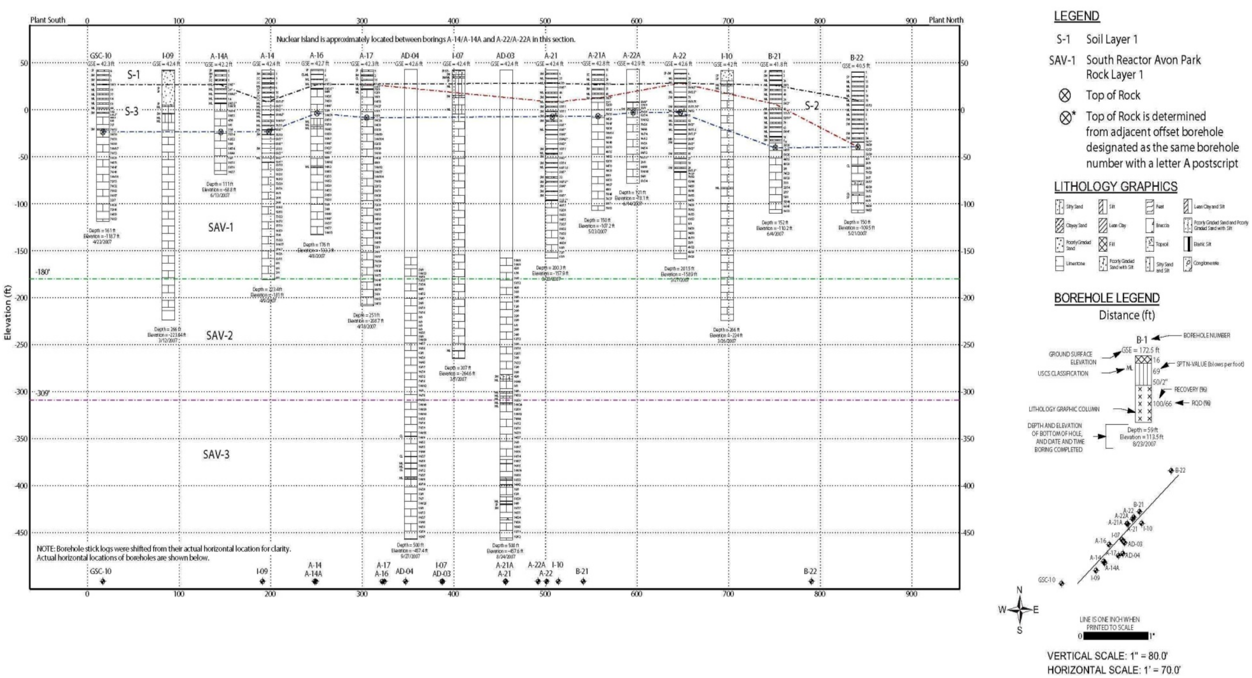


Figure 2.5.4-2. LNP 1 Plant North-South Cross-Section
(FSAR Figure 2.5.4.2-202A)

2.5.4.2.4 Geophysical Surveys

FSAR Section 2.5.4.4 discusses the scope, objectives, and results of the borehole geophysical survey methods performed at the LNP site.

2.5.4.2.4.1 Descriptions of Borehole Geophysical Surveys

FSAR Section 2.5.4.4.1 describes the seismic and non-seismic geophysical survey methods used to characterize the soil and rock properties. The applicant used two phases of suspension P-S velocity logging surveys as the primary data source to characterize the dynamic properties of soil and rock at the LNP site. The applicant also used the acoustic televiewer surveys to assess the verticality of all boreholes, obtain acoustic images of the borehole walls, and identify the dip and orientation of bedding planes and fractures. Downhole V_s surveys were completed to confirm the suspension P-S velocity logging survey results. The applicant also performed natural gamma, gamma-gamma, neutron-neutron, and induction surveys to acquire additional data to assist in the characterization of the subsurface. From this data, the applicant observed differences in soil and rock type, density, porosity, permeability, and pore fluid composition along the boring depth and between borings.

2.5.4.2.4.2 Geophysical Survey Investigation Results

FSAR Section 2.5.4.4.2 summarizes the results obtained from the various borehole geophysical surveys performed, including V_s and V_p profiles, lithological interpretations, and material property assessments.

2.5.4.2.4.3 Suspension P-S Velocity Logging Surveys

The following sections summarize FSAR Section 2.5.4.4.2.1, including the results obtained from the suspension P-S velocity logging surveys at the South, LNP Unit 1, and North, LNP Unit 2, sites.

LNP Unit 1 (South Reactor)

In the soil overburden above the top of rock, soil layers S-1, S-2 and S-3, the applicant measured V_s values between 380 to 1,410 meters per second (m/s) (1,250 fps to 4,630 fps) and observed a gradual transition from low V_s soil to high V_s rock at depths of 16.7 to 24.4 m (55 to 80 ft). The applicant identified three rock layers at LNP Unit 1: SAV-1 from top of rock down to an El. of -54.9 m (-180 ft); SAV-2 from an El. of -54.9 to -94.2 m (-180 to -309 ft); and SAV-3 from an El. of -94.2 to -139.6 m (-309 to -458 ft). SER Figure 2.5.4-3 shows the LNP Unit 1 East–West seismic profile obtained from the suspension P-S velocity logging surveys, seismic data interpreted from downhole seismic surveys, and other geotechnical data.

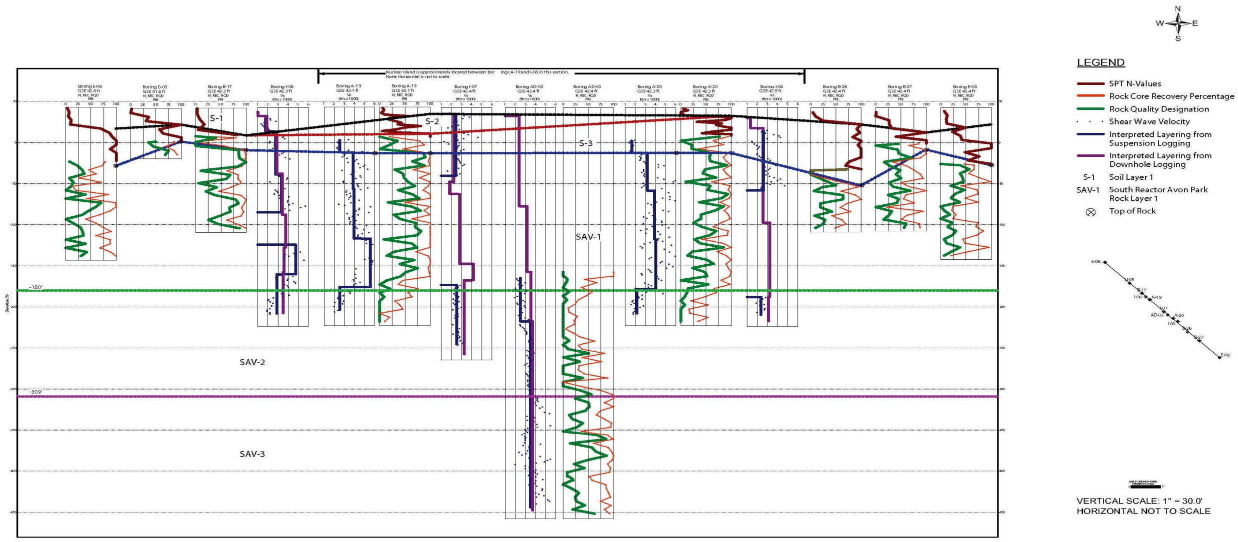


Figure 2.5.4-3. LNP1 East-West Shear Wave Velocity Profile
(FSAR Figure 2.5.4.2-204B)

<u>LNP Unit 2 (North Reactor)</u>

The applicant stated that for soils above top of rock the V_s ranges from 190 to 1,311 m/s (620 to 4,300 fps) with the transition from low V_s soil to high V_s rock at an approximate depth of 12 m (39.4 ft). The applicant identified four rock layers at LNP Unit 2: NAV-1 from top of rock down to an El. of -29.6 m (-97 ft); NAV-2 from an El. of -29.6 to -45.1 m (-97 to -148 ft); NAV-3 from an El. of -45.1 to -92.3 m (-148 to -303 ft); and NAV-4 from an El. of -92.3 m to -139.6 m (-303 to -458 ft). The applicant concluded that the suspension P-S velocity logging surveys in uncased boreholes below depths of 15.2 m (50 ft) produced good quality velocity profiles. However, the results obtained at shallower depths are inconsistent due to the presence of the borehole casing, and erosion and collapse of the borehole walls during drilling. The applicant observed that the rock V_s measured at LNP Unit 1 is lower than at LNP Unit 2, and noted a greater variation in V_s measurements in SAV-1 than in NAV-1 and NAV-2. SER Figure 2.5.4-4 shows an east to west geophysical and geological cross-section underlying Unit 2. The various soil and limestone layers are designated in this figure along with the measured V_s profiles.

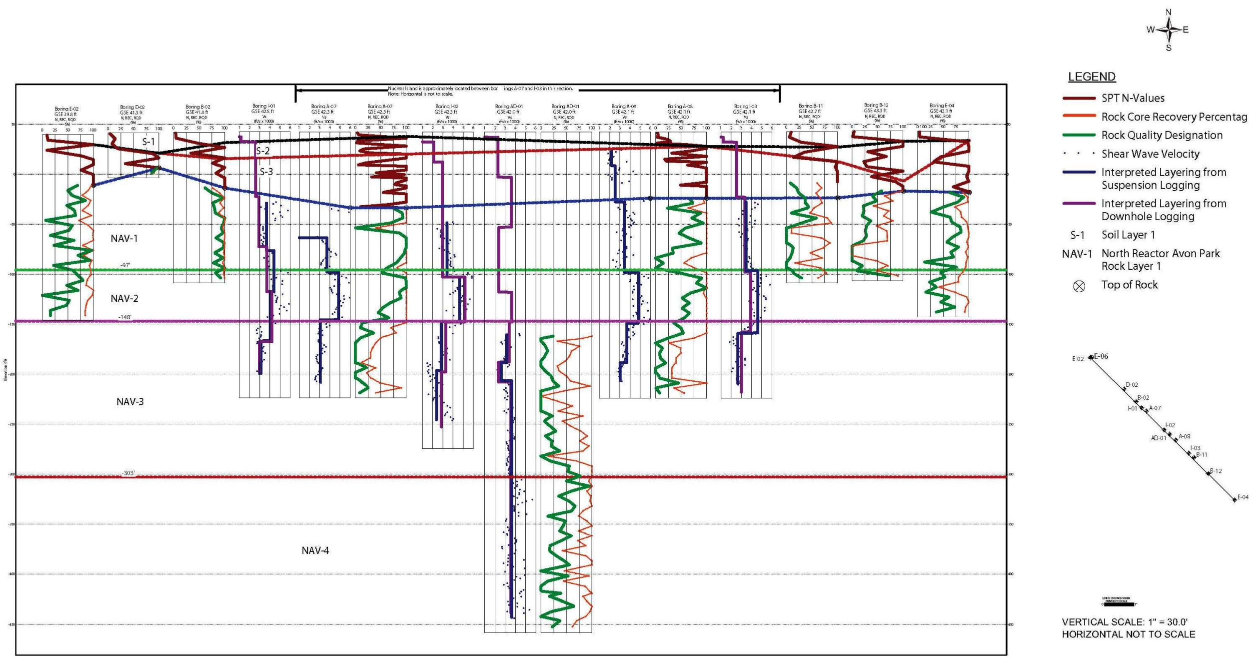


Figure 2.5.4-4. Subsurface Fence Diagram with V_s Results at LNP 2 Plant East to West (FSAR Figure 2.5.4.2-205b)

2.5.4.2.4.4 Acoustic Televiewer Surveys

The applicant used acoustic televiewer surveys to measure borehole deviation and image fractures, bedding planes, and eroded areas along the borehole walls. The applicant identified two vertical open fractures, which it considered a significant occurrence because of the rarity of intersecting vertically-oriented joints while drilling a vertically-oriented borehole.

2.5.4.2.4.5 Downhole Velocity Surveys

The applicant stated that the suspension P-S velocity logging method is more precise than the downhole method, and therefore used the results of the suspension P-S velocity logging in the engineering analyses and the downhole results for confirmation of the suspension P-S velocity logging results.

2.5.4.2.4.6 Natural Gamma Log

The applicant noted the increased clay content in the soil deposits above the top of rock at LNP Units 1 and 2, which was used as one marker in delineating the top of rock. The applicant indicated that a shallow, more weathered clayey profile exists at LNP Unit 1 than at LNP Unit 2. At LNP Unit 2, the applicant observed that the borings generally show a uniform higher natural gamma response with the exception of one borehole, which exhibited a response 1.5 times larger than the response found in the other borings. The applicant postulated the presence of more clay in this boring.

2.5.4.2.4.7 Gamma-Gamma (Density) Log

The applicant used these results to determine the presence of soil infill, which can be correlated to poor rock quality. However, the applicant was unable to correlate the gamma-gamma logs to the drilling logs, which may indicate that the low density zones identified in the gamma-gamma logs and the karst features reported in the core logs are limited in extent. The applicant concluded that all of the significant low density zones occur no deeper than 61 m (200 ft) below the existing ground surface.

2.5.4.2.4.8 Neutron-Neutron (Porosity) Log

The applicant stated that low neutron-neutron values indicate an increase in porosity and lower density, while higher values indicate a decrease in porosity and higher density. The applicant stated that the porosity is lower at LNP Unit 1 than at LNP Unit 2. The applicant identified a relatively lower porosity zone at depths between 42.6 and 57.9 m (140 and 190 ft) below the existing ground surface that is broader at LNP Unit 1 and more distinct at LNP Unit 2.

2.5.4.2.4.9 Induction (Conductivity) Log

The applicant related higher conductivity readings to increased clay content or pore fluids having an increased specific conductance. At LNP Unit 1, the applicant measured high conductivities between depths of 27.4 and 56.3 m (90 to 185 ft) below the existing ground surface, which it concluded were randomly distributed localized thin features. At LNP Unit 2, the applicant found that the conductivities in the upper 30.4 m (100 ft) are more uniform than those occurring at LNP Unit 1. A thin, high conductivity zone occurs in the LNP Unit 2 logs between depths of 27.4 to 28.9 m (90 to 95 ft).

2.5.4.2.4.10 Criteria for Use of Geophysical Survey Results as Design Parameters

The applicant used the suspension P-S velocity logging data as the primary source of V_s and V_p data for the engineering analyses. The applicant used acoustic televiewer, caliper and deviation survey data for borehole verticality checks, lithologic and stratigraphy determinations, and examinations of fractures. The non-seismic geophysical tools provided data that the applicant used to rule out continuity of voids from boring to boring.

2.5.4.2.5 Excavations and Backfill

FSAR Section 2.5.4.5 describes the applicant's plans for the excavation and backfill of the nuclear islands, including the planned diaphragm wall, excavation extents, and assumed properties of concrete backfill to be placed underneath and adjacent to the safety-related structures.

2.5.4.2.5.1 Diaphragm Walls and Grouting

FSAR Section 2.5.4.5.1 discusses the purpose of the diaphragm walls and grouting. The applicant stated that the diaphragm walls will serve as a temporary excavation support system to facilitate the excavation from the existing ground surface down to an El. of -7.3 m (-24 ft). The applicant noted that the diaphragm walls in combination with the grouted portion of the foundation, will aid construction dewatering by reducing seepage rates into the excavation. SER Figure 2.5.4-5 shows the extent of the excavation and diaphragm wall in the plan for LNP Unit 1. The excavation limits and diaphragm wall are coincident. SER Figure 2.5.4-6 shows a cross-section of the LNP Unit 1 diaphragm wall and grouting limits. Also shown on this figure is the RCC bridging mat and pier-supported seismic Category II and nonsafety-related structures that surround the nuclear island.

LNP COL 2.5-7

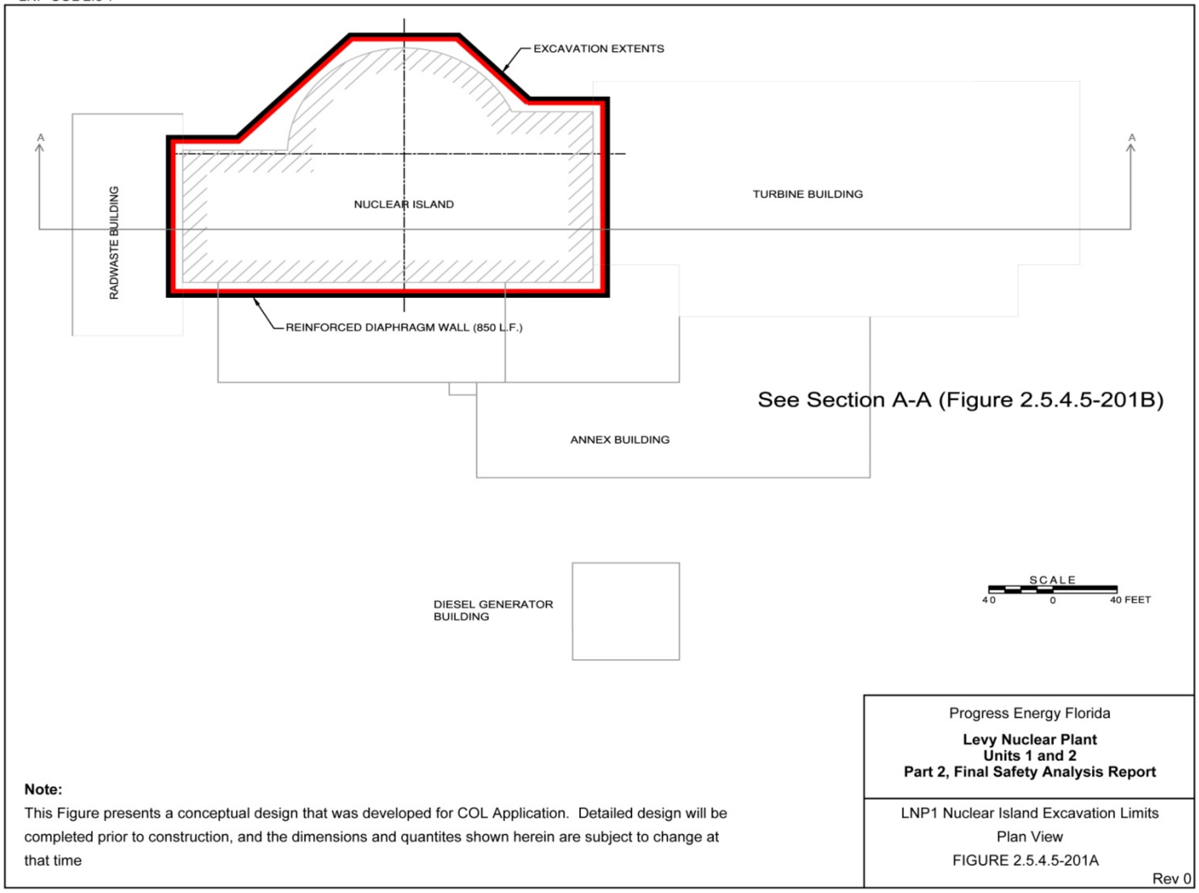


Figure 2.5.4-5. Plan View of LNP Unit 1
(FSAR Figure 2.5.4.5-201A)

2.5.4.2.5.1.1 Perimeter Diaphragm Wall

The diaphragm wall will be constructed prior to commencing excavation. The applicant stated that it will use hydrofraise equipment to excavate and key the diaphragm walls approximately 9.1 m (30 ft) into competent rock. The hydrofraise equipment consists of a crane hoisted drilling machine that cuts a vertical slot down to the desired depth through soil and rock. The wall is excavated in alternating panels that are initially supported by the drilling fluid and subsequently backfilled with tremie concrete. The wall will be tied-back by rows of pre-stressed anchors spaced 3 m (10 ft) on center. The anchor pull out resistance will be developed by grouting the anchors into the Avon Park limestone. The wall will be constructed of 1.06 m (3.5 ft) thick of concrete with compressive strength of 27.6 MPa (4,000 psi) and reinforced with a steel reinforcement cage. The diaphragm wall will serve as an excavation support system and vertical seepage barrier.

2.5.4.2.5.1.2 Permeation Grouting

In order to decrease the permeability of the uppermost layer of the Avon Park limestone, the applicant plans to inject grout from an El. of -7.3 to -30.1 m (-24 to -99 ft) within the limits of the diaphragm wall (see SER Figure 2.5.4-6) to fill voids associated with joint sets and bedding planes. Acting together, the diaphragm wall and the grouted limestone formation will form a "bathtub" and slow ground water seepage into the excavation for the foundation. The grouted section will also reduce the potential for future solution activity by cutting off flow paths. The applicant worked out the details of the methodology and materials planned for the production grouting during a grout test program, which is discussed below.

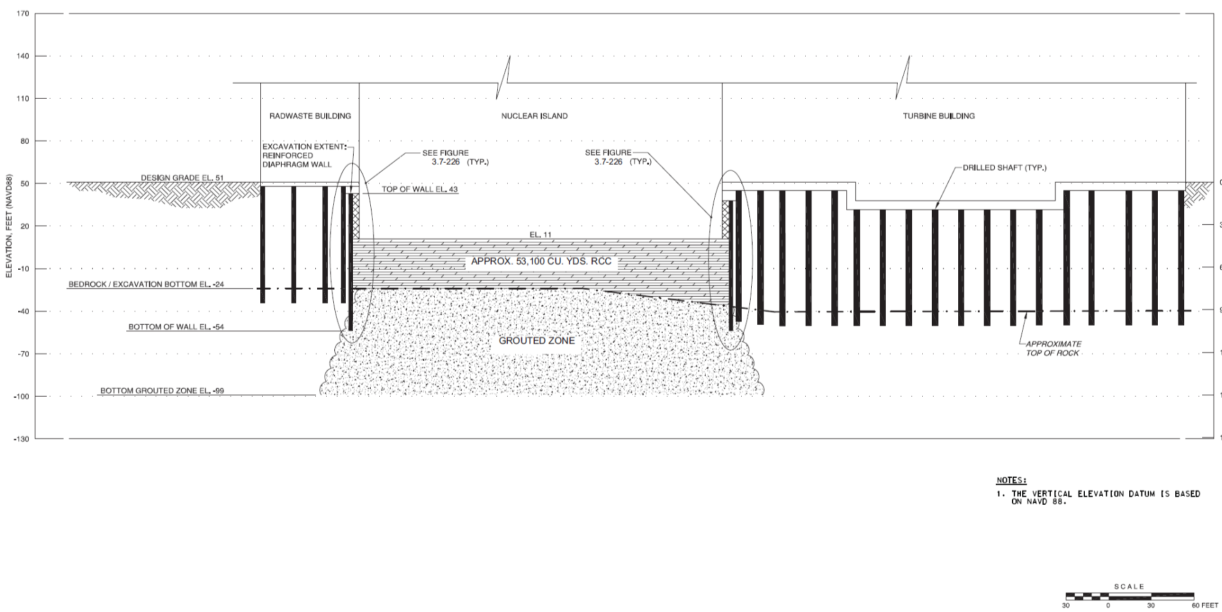


Figure 2.5.4-6. Profile View of LNP 1
(FSAR Figure 2.5.4.5-201B)

2.5.4.2.5.1.3 Grouting Method

The applicant stated that the primary method for the grouting operation will consist of grouting through boreholes, including angled boreholes, to intercept vertical joints. The applicant will perform the grouting using the upstage method with pneumatic packers when possible, and a suite of grout mixes of varying viscosities. Where necessary, the applicant will use downstage grouting methods to prevent borehole collapse. The applicant plans to space primary grout holes on 4.8 m (16 ft) centers and split space to 2.4 m (8 ft) centers. Decisions regarding the use of tertiary boreholes on 1.2 m (4 ft) centers will be determined during the production grouting program. Based on the results of the grout test program, the applicant established grouting pressures of 11.3 kilopascals (kPa) per meter (0.5 psi per foot) of depth during production grouting.

2.5.4.2.5.1.4 Grout Test Program

The applicant performed a grout test program to validate the grout mix design, grouting pressures and grouting techniques; measure any change in VS due to grouting; evaluate the permeability within the grouted zone; and determine the grout take prior to construction. The applicant grouted outside the footprint of the nuclear island, between 42.9 and 20.1 m (141 and 66 ft) below the surface primarily using vertical holes. This interval coincides with the intended grout zone during production. Using state-of-the-art monitoring equipment, the applicant determined best practice grouting pressures, grout mixes, and other grouting criteria. The results of the grout test program demonstrated that a reduction in rock mass permeability to reduce seepage into the excavation to acceptable limits was achieved and the V_S of the limestone was not appreciably affected by the presence of the grout.

2.5.4.2.5.2 Excavation Extents

FSAR Section 2.5.4.5.2 discusses the extent of the excavations, which are within the limits of the diaphragm walls. Outside the excavation for the nuclear island, the nonsafety-related structures will be supported on drilled shaft foundations socketed into rock (see SER Figure 2.5.4-6). The applicant plans a pilot hole at each drilled shaft location to determine the bearing depth at the base of the drilled shaft. The applicant also plans to excavate and replace the very loose to loose near surface soils to a depth of 2.13 m (7 ft) underlying the auxiliary buildings with engineered fill. SER Figures 2.5.4-5 and 2.5.4-6 show the conceptual plans for the excavation, diaphragm wall, grouting limits and seismic Category II and nonsafety-related structures surrounding the nuclear island.

2.5.4.2.5.3 Excavation Methods and Subgrade Improvement

FSAR Section 2.5.4.5.3 describes the methods for excavation and subgrade improvement. The applicant identified an El. of -7.3 m (-24 ft) as the depth at which materials with the most desirable properties for foundation stability were encountered. The in-situ rock at this elevation needs to be moderately to highly cemented, without solution features, loose rock or open or soil-filled joints or fractures. The applicant plans to remove and replace, or improve, foundation conditions that do not meet the design criteria. Excavation will be by ordinary means using earth moving equipment within the diaphragm walled area. The excavation will be incremental to allow geologic mapping and installation of anchors in the reinforced concrete diaphragm wall. Once the excavation reaches an El. of -7.3 m (-24 ft), the applicant will prepare the surface of the Avon Park limestone by removing loose rock from the surface and excavating soil from open joints. The applicant plans to use dental concrete as backfill in all open joints and as a leveling course for the RCC placement.

2.5.4.2.5.4 Properties of Backfill Beneath and Adjacent to the Nuclear Island

FSAR Section 2.5.4.5.4 discusses the backfill properties beneath and adjacent to the nuclear islands. The applicant plans to replace unsatisfactory soils with a 10.7 m (35 ft) thick RCC bridging mat bearing on the surface of the prepared Avon Park Formation. The applicant stated that the RCC bridging mat provided a uniform subgrade for the nuclear island mat foundation and the capability to bridge potential karst features. Between the diaphragm wall and nuclear island basemat, the applicant plans to use a concrete-like controlled low strength material (CLSM) as backfill. SER Figure 2.5.4-7 shows the location of the CLSM, and SER Table 2.5.4-3 summarizes the characteristics assumed for both the RCC and the CLSM. The applicant plans to develop further specifications for each backfill type and quality tests prior to construction.

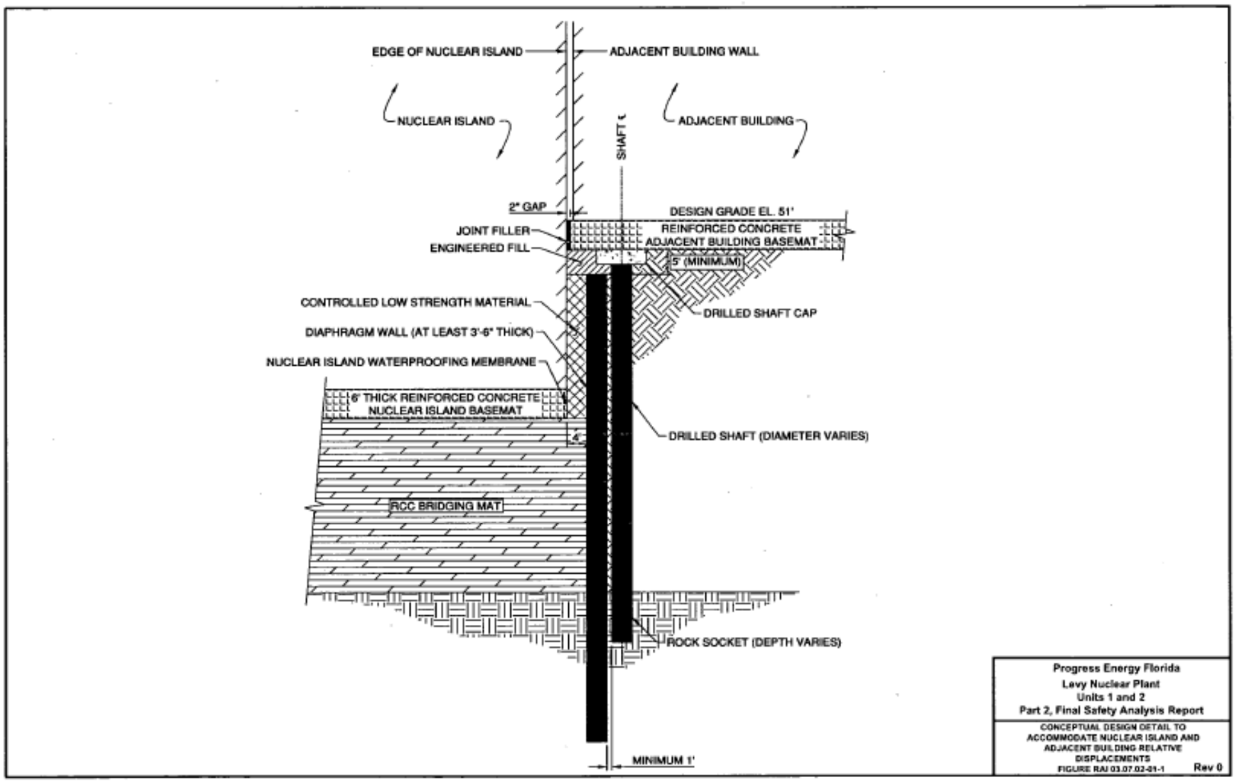


Figure 2.5.4-7. Detail Showing Location of CLSM Between Nuclear Island and Diaphragm Wall (RAI Figure 03.07.02-01-1)

Table 2.5.4-3. As-Built Engineering Properties of Backfill and Structural Fill (FSAR Table 2.5.4.5-201)

Backfill Type	Strength Parameters, MPa (psi)	V_s, m/s (fps)
RCC Bridging Mat	1-Year Compressive Strength: 17.2 (2,500)	1,066 (3,500)
CLSM Backfill	28-Day Compressive Strength: 3.4 (500)	304 ,000)

2.5.4.2.5.4.1 Roller Compacted Concrete Mat Test Pad

The applicant plans to construct a RCC test pad in order to define the material properties and develop the quality control requirements. The applicant stated that among the properties to be tested are mix design, material control testing, strength testing, concrete placement, and field-testing. The applicant also stated that the results of these tests will ensure that all material property specifications are met and the RCC test pad has the same specifications as in FSAR Section 2.5.4.5.4.

2.5.4.2.6 Ground Water Conditions

FSAR Section 2.5.4.6 summarizes the pre- and post-construction ground water elevations and the preliminary plans for construction dewatering. Also in this section, the applicant summarized the existing groundwater table, which ranges from 0.3 to 1.5 m (1 to 5 ft) below the existing ground surface. The applicant concluded that the post-construction ground water elevation at the LNP site is not expected to rise above an El. of 14.6 m (48 ft) below the final site grade at an El. of 15.5 m (51 ft). The applicant referred to FSAR Section 2.4.12.5 for additional details on the groundwater conditions at the site.

2.5.4.2.6.1 Construction Dewatering

FSAR Section 2.5.4.6.2 discusses the estimated construction dewatering flow rates and dewatering methods for LNP Units 1 and 2. The applicant determined that the diaphragm walls will minimize the lateral ground water flow while grouting of the Avon Park Formation will minimize upward seepage and resist uplift pressure. The applicant used MODFLOW 2000 to model the proposed excavation and observe the expected upward gradients and ground water flow rates into the excavation.

To account for variations in the effectiveness of the grout, the applicant varied the gross permeability of the grouted sections in the model. Permeability of the ungrouted sections was based on hydraulic conductivity field tests. The applicant plans to evaluate the exposed subgrade rock and eliminate any significant leakage through a second round of grouting. The applicant also plans to employ a ground water monitoring program during construction to measure the head differential inside and outside of the diaphragm walls and the uplift pressure across the bottom of the excavation.

2.5.4.2.7 Response of Soil and Rock to Dynamic Loading

FSAR Section 2.5.4.7 summarizes the response to dynamic loading for both soil and rock at the LNP site. Because ground motions at the site are low, the applicant did not perform dynamic triaxial shear tests or resonant column torsional shear tests but instead accounted for any uncertainty in modulus and damping relationships by assuming a range of behaviors for the softer layers using two sets of EPRI curves for the site response analysis. The applicant also stated that the potential for non-tectonic deformation is negligible.

2.5.4.2.8 Liquefaction Potential

FSAR Section 2.5.4.8 discusses the potential for liquefaction at the LNP site. The applicant computed the factor of safety (FS) against liquefaction generated by the SSE following the guidance provided in RG 1.198, which recommends using the method of analysis described by Youd et al. (2001).

2.5.4.2.8.1 Soil and Ground Water Conditions

FSAR Section 2.5.4.8.1 discusses the soil conditions at LNP Units 1 and 2 at the time of exploration and employed in the liquefaction analysis. The soil profile consists of loose to very dense sands and silts and some clay overlying the Avon Park Formation. The applicant noted that ground water is typically within 0.9 m (3 ft) of the existing ground surface; the existing ground surface being at approximately an El. of 13.1 m (43 ft). The applicant noted that liquefaction below the nuclear island is not possible as the nuclear island will be founded on RCC overlying the Avon Park limestone. Because the soils outside the diaphragm wall are potentially liquefiable, the applicant included them in the liquefaction analysis, with the exception of the top 2.1 m (7 ft) of soils, which will either be removed or improved as described previously in Section 2.5.4.2.5.3 of this SER.

2.5.4.2.8.2 Liquefaction Analysis Procedure

FSAR Section 2.5.4.8.2 describes the liquefaction analysis procedure, specifically the calculation of the factor of safety against liquefaction, which is a function of cyclic stress generated by the SSE compared to the dynamic strength of the soils. In accordance with RG 1.198, the applicant considered cohesionless soils with FS less than or equal to 1.1 liquefiable, and soils with FS greater than 1.4 to be non-liquefiable. For soils with FS in the intermediate range, greater than 1.1 but less than 1.4, the applicant considered the deleterious effect of increased dynamic pore pressures on the strength of the soil.

2.5.4.2.8.3 Results of Liquefaction Analysis

FSAR Section 2.5.4.8.5 discusses the results of the liquefaction analyses, which show that some random near surface soils beyond the limits of the nuclear island may experience liquefaction. The applicant stated that the presence of random liquefied zones outside of the nuclear island would not interfere with the AP1000's basemat stability with regard to sliding. The applicant based this conclusion on the fact that the liquefied zones are either isolated, negligible, outside the zone that provides resistance to sliding, or will be excavated and replaced with non-liquefiable material. In addition, the applicant stated that the earthwork design incorporates vertical and horizontal drains to prevent buildup of excess pore pressures that cause liquefaction. The applicant also stated that the design of the drilled piers will account for the random liquefied zones such that the lateral stability of the drilled piers will not be affected. The drilled piers support the seismic Category II and nonsafety-related structures and are reviewed in LNP SER Sections 3.7 and 3.8.5.

2.5.4.2.9 Earthquake Site Characteristics

In FSAR Section 2.5.4.9, the applicant referred to FSAR Sections 2.5.2.5 and 2.5.2.6 for a discussion of the methods used to calculate the site amplification at the GMRS elevation and the FIRS.

2.5.4.2.10 Static Stability

FSAR Section 2.5.4.10 discusses the analyses performed to assess the foundation bearing capacity, sliding, foundation settlement, and lateral pressures against below-grade walls.

2.5.4.2.10.1 Bearing Capacity

FSAR Section 2.5.4.10.1 states that the bearing capacities obtained under static and dynamic loading conditions satisfy the safety requirements set forth in the AP1000 DCD.

2.5.4.2.10.1.1 Bearing Capacity Analysis Methodology

The applicant stated that the critical subsurface bearing material beneath each nuclear island is the RCC bridging mat. The applicant used the permissible service load stress equation from American Concrete Institute (ACI) 318-89 to determine the ultimate bearing stresses in concrete, and compared the bearing capacity of the RCC bridging mat to the bearing demand. The applicant determined FSs of 12.1 and 4.5 for the static and dynamic cases, respectively. The applicant noted that the factor of safety for the dynamic case was based on the dynamic bearing demand of 1.15 MPa (24 ksf) which envelops maximum bearing pressure of 0.97 MPa (20.29 ksf) from site-specify SSI analysis with the LNP site-specific SSE of 0.1g. The applicant concluded that the factors of safety are greater than 2.8 for the dynamic case. The applicant also indicated that the factors of safety are greater than the industry accepted factors of safety of 3 for the static case and 2 for the dynamic case.

The applicant used two procedures to determine the bearing capacity of the Avon Park limestone that supports the RCC bridging mat, the simplified American Association of State Highway and Transportation Officials (AASHTO, 2002) formulation for footings on broken or jointed rock, and the U.S. Army Corps of Engineers (USACE) formulation (USACE, 1992) for two different failure modes of rock. Additionally, the applicant used a 3D finite element method (FEM) analysis to compute the FS against bearing capacity considering the presence of postulated voids of different sizes at varied elevations and locations within the Avon Park limestone. The applicant determined that the FS against bearing capacity in the Avon Park limestone was at least 3 for the static case and 2.0 for the dynamic case.

2.5.4.2.10.2 Resistance to Sliding

FSAR Section 2.5.4.10.2 discusses the resistance of the nuclear islands to sliding. The applicant stated that it will found the RCC on Avon Park limestone that meets the design criteria and is clean of any loose material in order to achieve interlocking between the RCC bridging mat and the underlying rock. The applicant assumed zero adhesion and a friction angle of 48 to

60 degrees between the RCC bridging mat and underlying limestone, which is greater than the 35 degrees required by the AP1000 DCD.

2.5.4.2.10.3 Settlement

FSAR Section 2.5.4.10.3 discusses the settlement analyses performed for the LNP site. The applicant calculated small total and differential settlements that fall within the limits specified in the AP1000 DCD. Based on the settlement analyses, the applicant concluded that it satisfied all design criteria for foundation settlement at LNP Units 1 and 2.

2.5.4.2.10.3.1 Elastic (Total) Settlement under Foundation Loads

The applicant calculated the elastic settlement of the nuclear islands at LNP Units 1 and 2 based on the elastic properties of the Avon Park rock mass and obtained results from three methods: a 3D FEM analysis, AASHTO (2002), and elastic theory. The applicant stated that the average settlements obtained from the FEM analysis as measured at the base of the RCC bridging mat were 0.53 and 0.45 cm (0.21 and 0.18 in) at LNP Units 1 and 2, respectively. The other methods used were in agreement with the FEM analysis. The applicant stated that total settlements are likely to occur during construction, and noted that the AP1000 DCD settlement criterion is 7.6 cm (3 in).

2.5.4.2.10.3.2 Differential Settlement

Based on the settlement analysis results, the applicant determined that the maximum settlement occurs at the center of the nuclear island, and calculated a tilt of less than 1:1,200. The applicant concluded that the tilt was within the permissible differential settlement requirements of 1:1200 (1.27 cm in 15.24 m (0.5 inch in 50 ft)) allowed by the AP1000 DCD. Because the nonsafety-related buildings will be founded on drilled shafts socketed into competent rock, the applicant stated that the differential settlements between the nuclear island and the adjacent nonsafety-related buildings are negligible. The applicant planned to perform detailed settlement analyses for the surrounding nonsafety-related buildings prior to construction.

2.5.4.2.10.3.3 Subsurface Instrumentation

The applicant stated that it would monitor water levels and settlement (heave) during construction. As part of this monitoring program, the applicant stated that it will install piezometers outside the perimeter of the diaphragm walls at an El. of -7.3 m (-24 ft); and within the excavation at an El. of 0 and -8.8 m (0 and -29 ft); and below the grouted zone at an El. of -30.1 m (-99 ft).

The applicant stated that it will place settlement monitoring points at the four corners of each nuclear island and at the northernmost point of the containment building, and monitor these benchmarks before and during construction of the nuclear island basemat and sidewalls. The applicant also committed to install and monitor additional settlement points connected to the sidewalls of the nuclear islands 0.9 m (3 ft) above site grade during backfilling operations. Additionally, the applicant committed to monitor settlement after construction of the nuclear -

island until 90 percent of the expected settlement occurred. The applicant committed to establish a post-construction long-term settlement monitoring program using the settlement points established during construction.

2.5.4.2.10.4 Lateral Earth Pressures

FSAR Section 2.5.4.10.3.5 discusses the static and dynamic lateral earth pressures acting on the below-grade nuclear island sidewalls. The applicant considered the ground surface live load, crane load, pseudostatic earthquake load, hydrostatic pressure due to the water table, soil and CLSM backfill loads, and the strength of the backfill in its analysis of the lateral pressures on the nuclear island sidewalls. To minimize the soil stresses against the wall, the applicant plans to use hand-operated compaction equipment in areas adjacent to the nuclear island sidewalls. The applicant did not include the loads from adjacent structures in the lateral pressure calculation because these structures are supported by drilled piers socketed into rock.

2.5.4.2.11 Design Criteria

FSAR Section 2.5.4.11 summarizes the design criteria and methods used in the different analyses, including assumptions, and FS. The applicant compared the site-specific characteristics of bearing capacity, V_S, lateral variability and liquefaction potential to AP1000 DCD site criteria. Based on this comparison, the applicant concluded that the LNP site meets the AP1000 DCD site criteria.

2.5.4.2.12 Techniques to Improve Subsurface Conditions

FSAR Section 2.5.4.12 summarizes techniques the applicant proposed to improve subsurface conditions. To reduce the rock mass porosity and control ground water during excavation for the foundation, the applicant plans to grout the Avon Park limestone from an El. of -7.3 m (-24 ft) down to -32 m (-99 ft). The subsequent placement of a diaphragm wall penetrating 9.1 m (30 ft) into the Avon Park limestone will create a semi-impervious barrier around and below the area to be excavated for the placement of the RCC bridging mat. After dewatering the site, the applicant plans to incrementally excavate down to the Avon Park limestone at an El. of -7.3 m (-24 ft). The bottom surface of the excavation will be prepared for RCC placement by removing any loose rock or unsuitable foundation materials, and backfilling voids in the subgrade with dental concrete to level the surface. The prepared surface will receive the 10.7 m (35 ft) thick RCC bridging mat, which tops out at an El. of 3.3 m (11 ft). SER Figure 2.5.4-6 shows the East-West cross-section of LNP Unit 1 with the location of the diaphragm walls, RCC bridging mat and grouting limits.

2.5.4.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

The applicable regulatory requirements for the stability of subsurface materials and foundations are as follows:

- 10 CFR Part 50, Appendix A, GDC 2, "Design Bases for Protection Against Natural Phenomena," relates to the consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," applies to the design of nuclear power plant SSCs important to safety to withstand the effects of earthquakes.

- 10 CFR 100.23, provides the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and design of nuclear power plants.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the stability of subsurface materials and foundations are given in Section 2.5.4 of NUREG-0800.

- Geologic Features: In meeting the requirements of 10 CFR Parts 50 and 100, the section defining geologic features is acceptable if the discussions, maps, and profiles of the site stratigraphy, lithology, structural geology, geologic history, and engineering geology are complete and are supported by site investigations that are sufficiently detailed to obtain an unambiguous representation of the geology.

- Properties of Subsurface Materials: In meeting the requirements of 10 CFR Parts 50 and 100, the description of properties of underlying materials is considered acceptable if state-of-the-art methods are used to determine the static and dynamic engineering properties of all foundation soils and rocks in the site area.

- Foundation Interfaces: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of the relationship of foundations and underlying materials is acceptable if it includes: (1) a plot plan or plans showing the locations of all site explorations, such as borings, trenches, seismic lines, piezometers, geologic profiles, and excavations with the locations of the safety-related facilities superimposed thereon; (2) profiles illustrating the detailed relationship of the foundations of all seismic Category I and other safety-related facilities to the subsurface materials; (3) logs of core borings and test pits; and (4) logs and maps of exploratory trenches in the COL application.

- Geophysical Surveys. In meeting the requirements of 10 CFR 100.23, the presentation of the dynamic characteristics of soil or rock is acceptable if geophysical investigations have been performed at the site and the results obtained are presented in detail.

- Excavation and Backfill: In meeting the requirements of 10 CFR Part 50, the presentation of the data concerning excavation, backfill, and earthwork analyses is acceptable if: (1) the sources and quantities of backfill and borrow are identified and are shown to have been adequately investigated by borings, pits, and laboratory property and strength testing (dynamic and static) and these data are included, interpreted, and summarized; (2) the extent (horizontally and vertically) of all seismic Category I excavations, fills, and slopes are clearly shown on plot plans and profiles; (3) compaction specifications and embankment and foundation designs are justified by field and laboratory tests and analyses to ensure stability and reliable performance; (4) the impact of compaction methods are incorporated into the structural design of the plant facilities; (5) quality control methods are discussed and the QA program described and referenced; (6) control of ground water during excavation to preclude degradation of foundation materials and properties is described and referenced.

- Ground Water Conditions: In meeting the requirements of 10 CFR Parts 50 and 100, the analysis of ground water conditions is acceptable if the following are included in this section or cross-referenced to the appropriate sections in Section 2.4 of the FSAR: (1) discussion of critical cases of ground water conditions relative to the foundation settlement and stability of the safety-related facilities of the nuclear power plant; (2) plans for dewatering during construction and the impact of the dewatering on temporary and permanent structures; (3) analysis and interpretation of seepage and potential piping conditions during construction; (4) records of field and laboratory permeability tests as well as dewatering induced settlements; (5) history of ground water fluctuations as determined by periodic monitoring of 16 local wells and piezometers.

- Response of Soil and Rock to Dynamic Loading: In meeting the requirements of 10 CFR Parts 50 and 100, descriptions of the response of soil and rock to dynamic loading are acceptable if: (1) an investigation has been conducted and discussed to determine the effects of prior earthquakes on the soils and rocks in the vicinity of the site; (2) field seismic surveys (surface refraction and reflection and in-hole and cross-hole seismic explorations) have been accomplished and the data presented and interpreted to develop bounding P and S wave velocity profiles; (3) dynamic tests have been performed in the laboratory on undisturbed samples of the foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysteretic damping properties of the soils and the results included.

- Liquefaction Potential: In meeting the requirements of 10 CFR Parts 50 and 100, if the foundation materials at the site adjacent to and under seismic Category I structures and facilities are saturated soils and the water table is above bedrock, then an analysis of the liquefaction potential at the site is required.

- Static Stability. In meeting the requirements of 10 CFR Parts 50 and 100, the discussions of static analyses are acceptable if the stability of all safety-related facilities has been analyzed from a static stability standpoint including bearing capacity, rebound, settlement, and differential settlements under deadloads of fills and plant facilities, and lateral loading conditions.

- Design Criteria: In meeting the requirements of 10 CFR Part 50, the discussion of criteria and design methods is acceptable if the criteria used for the design, the design methods employed, and the factors of safety obtained in the design analyses are described and a list of references presented.

- Techniques to Improve Subsurface Conditions: In meeting the requirements of 10 CFR Part 50, the discussion of techniques to improve subsurface conditions is acceptable if plans, summaries of specifications, and methods of quality control are described for all techniques to be used to improve foundation conditions (such as grouting, vibroflotation, dental work, rock bolting, or anchors).

In addition, the geologic characteristics should be consistent with appropriate sections from: RG 1.28, “Quality Assurance Program Requirements (Design and Construction)” Revision 4; RG 1.132, Revision 2; RG 1.138, Revision 2; RG 1.198; RG 1.206; and RG 1.208.

2.5.4.4 *Technical Evaluation*

The NRC staff reviewed Section 2.5.4 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of information presented in the FSAR and the DCD completely represents the required information related to the stability of subsurface materials and foundations. The staff’s review confirmed that information contained in the application or incorporated by reference addresses the information required for this review topic. NUREG-1793 and its supplements document the results of the staff’s evaluation of the information incorporated by reference into the LNP COL application.

This SER section presents the staff’s evaluation of the geologic and geotechnical engineering information the applicant submitted in LNP COL FSAR Section 2.5.4 to address the stability of the subsurface materials and foundations at the LNP site and to resolve LNP COL Information Items 2.5-5 through 2.5-13, LNP COL 2.5-16. The staff’s evaluation of LNP COL 2.5-17 is addressed in Sections 3.8 and 14.3 of this SER. The technical information presented in LNP COL FSAR Section 2.5.4 resulted from the applicant’s surface and subsurface geologic and geophysical investigations performed within the site area. Through its review of LNP COL FSAR Section 2.5.4, the staff determined whether the applicant complied with the applicable regulations and conducted its investigations at an appropriate level of detail in accordance with RG 1.132, Revision 2, and RG 1.138, Revision 2.

To thoroughly evaluate the geologic, seismic and geophysical information the applicant presented, the staff obtained the assistance of geotechnical engineers at Information Systems Laboratory, Inc. (ISL) and the USACE. The staff, and its ISL and USACE contractors, visited the LNP site to review and confirm the interpretations, assumptions, calculations and conclusions the applicant presented related to the stability of subsurface materials and foundations at the LNP site.

In addition to the RAIs discussed below, which address specific technical issues related to the stability of subsurface materials and foundations of the LNP site, the staff asked several RAIs requesting clarifications and editorial corrections of figures and text associated with FSAR Section 2.5.4. The staff does not discuss these RAIs as part of its technical evaluation.

AP1000 COL Information Items

- LNP COL 2.5-5, LNP COL 2.5-6, LNP COL 2.5-7, LNP COL 2.5-8, LNP COL 2.5-9, LNP COL 2.5-10, LNP COL 2.5-11, LNP COL 2.5-12, LNP COL 2.5-13, and LNP COL 2.5-16

The staff's review of the information in LNP COL FSAR Section 2.5.4 to ensure that the COL information items were addressed satisfactorily is discussed below.

2.5.4.4.1 Geologic Features

The staff reviewed the summary of the regional and site geologic conditions, particularly the hazards that may affect the LNP site, provided in FSAR Section 2.5.4.1 as well as the description and characterization of the regional and site geology in FSAR Section 2.5.1. Section 2.5.1.4 of this SER includes the staff's technical evaluation of the regional and site geologic information. Based on the information and findings provided in FSAR Sections 2.5.4.1, 2.5.1 and 2.5.3, the staff concludes that the applicant provided adequate information regarding the geologic features at the LNP site. The detailed evaluation and staff findings with respect to the geologic features are provided in Sections 2.5.1.4 and 2.5.3.4 of this SER.

2.5.4.4.2 Properties of Subsurface Materials

The staff focused its review of LNP COL FSAR Section 2.5.4.2 on the applicant's description of the static and dynamic engineering properties of the soil and rock strata underlying the LNP site, and the methods used to determine the site engineering properties. The staff reviewed the applicant's field investigation methods and laboratory testing program used to determine the properties of the subsurface materials. The review was carried out with respect to the guidance of RG 1.132, Revision 2; RG 1.138, Revision 2; RG 1.208; and NUREG-0800 Section 2.5.4.

As stated in FSAR Section 2.5.4.1.2.1.4, both LNP nuclear islands will be supported by a 10.6 m (35 ft) thick RCC bridging mat, which will replace unsatisfactory weathered limestone between an El. of 3.35 and -7.3 m (11 and -24 ft). The RCC bridging mat will be supported by the underlying Avon Park limestone beginning at an El. of -7.3 m (-24 ft). The bearing capacity of the Avon Park limestone depends on the rock mass strength parameters, which are a function of the geologic strength index (GSI), material constant (m_i), E_{rm}, and elastic modulus reduction factor. The staff focused its review on the derivation of these material parameters to verify that the strength parameters used in the applicant's engineering analyses were conservative.

2.5.4.4.2.1 Geological Strength Index (GSI)

The staff reviewed the derivation of the GSI, an indicator of the rock mass strength and structural integrity. In RAI 2.5.4-7a, the staff asked the applicant to describe how it determined the GSI. The staff also asked the applicant to discuss how it factored joint sets, bedding planes, and low or no recovery zones into the GSI determination.

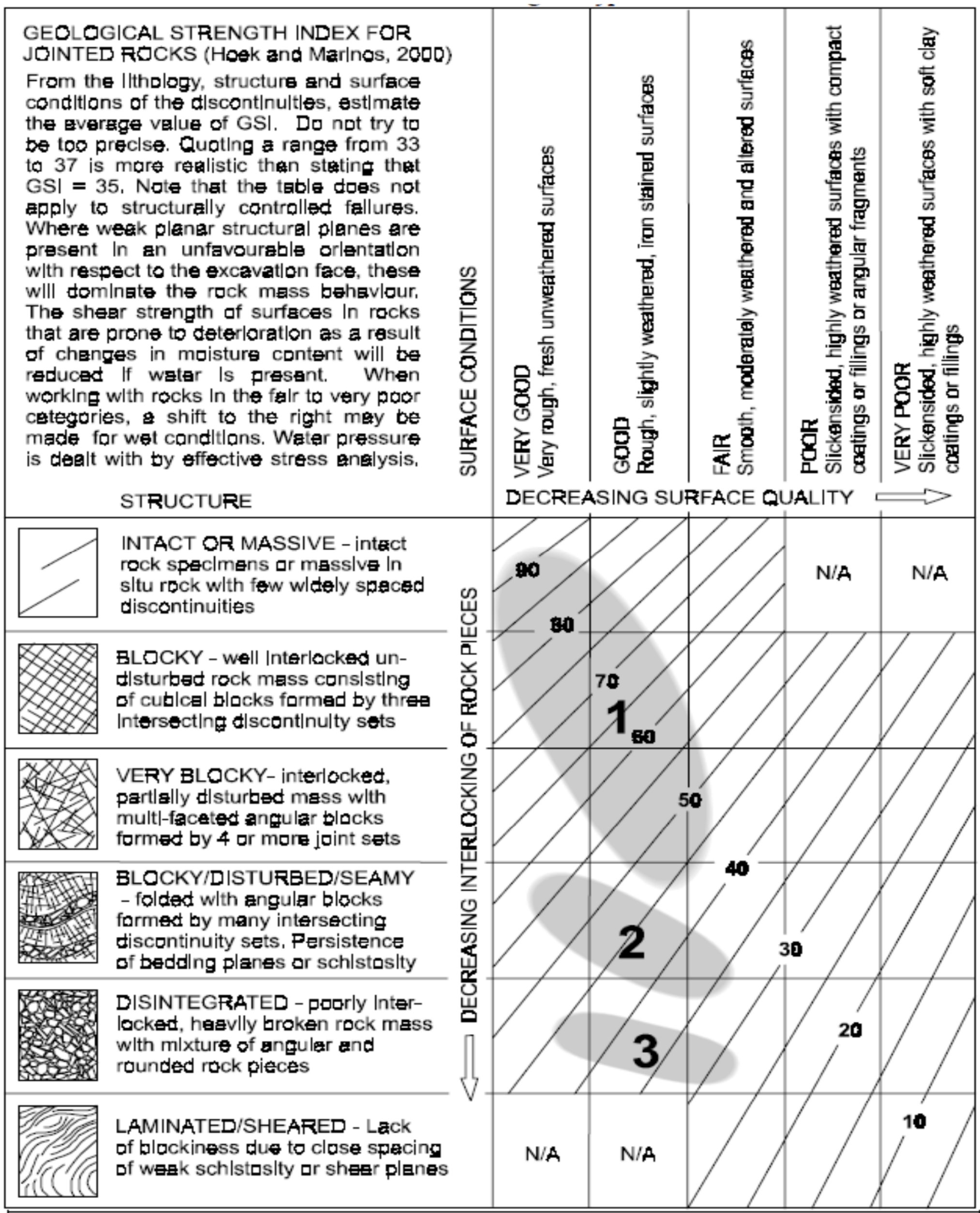
In its April 2, 2009, response, the applicant stated that for every core run, it obtained the rock mass rating (RMR) using the systems proposed by Bieniawski (1989) and Robertson (1988). To estimate the GSI, the applicant used the correlation between RMR and GSI developed by Hoek and Brown (1997) which explicitly considers joint sets and bedding planes in its determination of GSI. Specifically, the discontinuity spacing, discontinuity conditions, and orientation of the discontinuities are integral to the calculation of GSI. To account for the presence of weaker materials not recovered, the applicant applied reductions in the measured strength to those rock cores that exhibited low recovery rates. The applicant concluded that because it obtained GSI values from an extensive dataset consisting of every core run at the LNP site, and conservatively considered the no recovery zones, its determination resulted in lower-bound GSI values. The applicant subsequently used these lower-bound GSI values to determine conservative rock mass strength properties for the bearing capacity sensitivity analyses discussed in this SER Section 2.5.4.4.10. The applicant concluded that the input parameters are conservative.

In its response to RAI 2.5.4-7a, the applicant also stated its intent to gather additional data in order to evaluate the properties of materials, which were not recovered during core drilling. In a January 19, 2010, supplemental response to RAI 2.5.4-7a, the applicant stated that, based on the results of the offset boring program discussed in detail in Section 2.5.4.4.4.3, the rock mass property analysis, including the determination of GSI, is conservative.

The staff reviewed the applicant’s response to RAI 2.5.4-7, the RMR systems presented in the USACE Engineering Manual 1110-1-2908, and the GSI rating criteria presented in the Hoek-Brown method as described in Marinos and Hoek (2000). Based on the Hoek-Brown state-of-the-art method, the staff concludes that the estimated GSI is reliable because it considers joint sets and bedding planes, the condition of the discontinuities and the orientation of the discontinuities. In considering zones where core drilling did not recover rock cores, the applicant reduced the strength of the intact cores to account for the missing information in its determination of the GSI. Because the applicant later determined through the offset drilling program that the “no recovery” zones were weathered-in-place Avon Park limestone, and not voids or soil infill, the staff concludes that the applicant conservatively accounted for the presence of weaker materials. This conclusion is supported by the range in V_S measurements made in the no recovery zones, which are the same as zones where core was recovered. Accordingly, the staff concludes that the Hoek–Brown method of determining the GSI used by the applicant as described by Marinos and Hoek (2000) is acceptable.

Based on its review of the results of the offset boring program, the staff also noted that the presence of weathered limestone in the offset borings yields three very important conclusions:

(1) the no recovery zones indicated in the borings are not karst features; (2) the elastic modulus as derived from the V_s measurements is reliable; and (3) the GSI is conservatively determined. To demonstrate that the GSI is conservative, the staff consulted SER Figure 2.5.4-8, which presents a typical GSI range for limestone from Marinos and Hoek (2000).



*WARNING:
The shaded areas are indicative and may not be appropriate for site specific design purposes.
Mean values are not suggested for indicative characterisation; the use of ranges is recommended

1. Massive
2. Thin bedded
3. Brecciated

Figure 2.5.4-8. Typical GSI Factors for Limestone
(Modified from Marinos and Hoek, 2000)

In SER Figure 2.5.4-8, Marinos and Hoek (2000) show the typical limestone GSI values in the shaded zones labeled 1, 2 and 3, which range from 28 to 75 for disintegrated to blocky limestone, with fair to good discontinuity surface quality. The staff noted that most of the Avon Park Formation at the LNP site would fall in this range, with the exception of the severely weathered Avon Park limestone recovered at bedding planes and eroded vertical joints. The staff then overlaid the applicant's estimated GSI range on this figure, shown as labeled, and observed that the applicant's estimated GSI range of 21 to 38 corresponds to a disintegrated to blocky limestone with discontinuity surface quality that would be described as good to very poor. Based on this information as well as its review of the borings and other field data, the staff concludes that this is a conservative representation of the Avon Park limestone. The staff also compared these values with typical limestone GSI values, shaded areas 1, 2 and 3 on SER Figure 2.5.4-8, and concludes that the applicant's estimations of GSI values are conservative. Thus, the staff concludes that the applicant's estimated GSI values adequately represent the observed structure of the Avon Park limestone. Accordingly, RAI 2.5.4-7a is resolved.

2.5.4.4.2.2 Material Constant (m_i) Value

Because the m_i value is a material constant also used as input to the Hoek-Brown failure criteria to determine the shear strength of the rock mass, in RAI 2.5.4-14, the staff asked the applicant to justify its selection of a m_i value of 8.

In its June 8, 2009, response to RAI 2.5.4-14, the applicant stated that the recommended values of m_i for micritic limestone evolved from 8 (Hoek and Brown, 1997) to 9±2 (Marinos and Hoek, 2000) to 8±3 (RocLab 1.031, 2007). The applicant also stated that Marinos and Hoek (2000) include m_i values of 9±3 for dolomite. The applicant concluded that because micritic limestone represents the lower bound carbonate limestone m_i value the selected value of 8 is conservative.

In order to confirm the applicant's m_i estimate, the staff reviewed Marinos and Hoek (2000) and considered the published m_i values of 9±2 for micritic limestone and 9±3 for dolomite. Because much of the Avon Park limestone has been dolomitized, the staff notes that the selection of 8 represents the lower bound as shown in SER Table 2.5.4-4. Because the m_i value is a measure of the frictional properties of intact rock, the staff also considered the relationship between GSI, friction angle and m_i shown in SER Figure 2.5.4-9 for additional evidence that this m_i value is conservative. SER Figure 2.5.4-9 shows that for the range of GSI of 20 to 40 determined for the LNP site, and a conservative assumption of friction angle equal to 30 degrees, the estimated m_i would be in the range of 11 or greater. Therefore the staff concludes that the m_i value that the applicant selected is in the lower bound of the frictional strength of the Avon Park limestone. Because this value is based on the most recently published m_i estimate for micrite (RocLab 1.031, 2007), the staff concludes that the m_i value of 8 is both reasonable and conservative for the LNP site. Accordingly, RAI 2.5.4-14 is resolved.

Table 2.5.4-4. (from Marinos and Hoek, 2000)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerates * Breccias *	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2

* indeterminate range of values

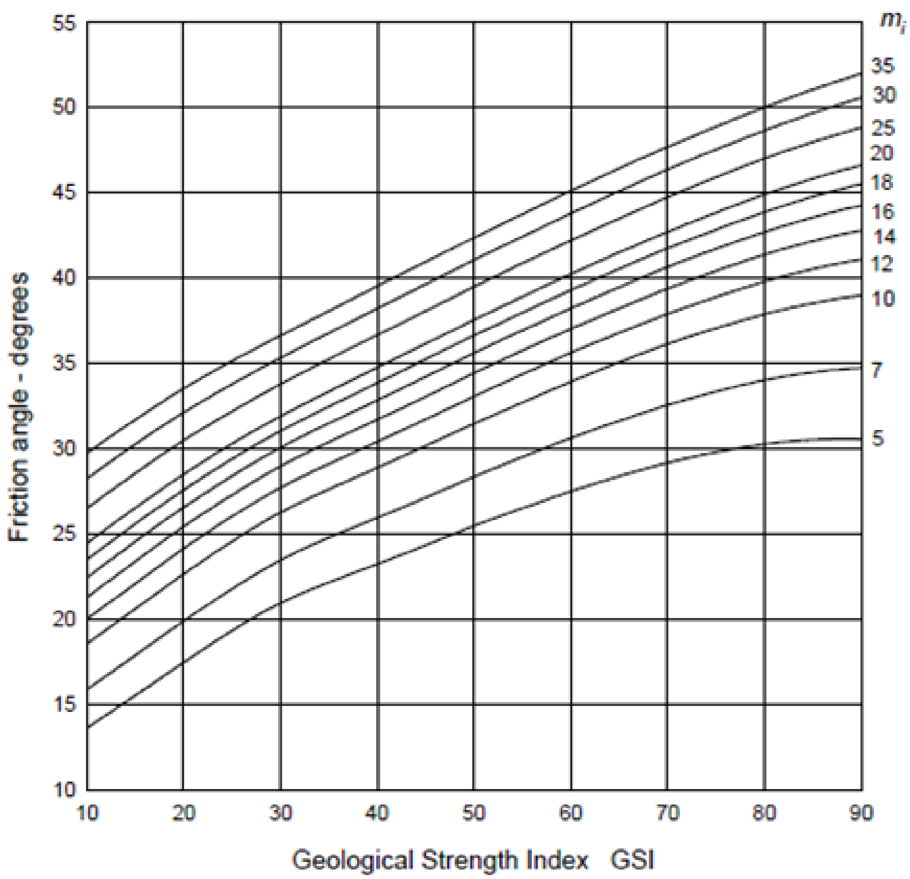


Figure 2. Friction angle φ for different GSI and m_i values, for depths more than 30m.

Figure 2.5.4-9. Friction Angle for Different GSI and m_i Values (from Marinos and Hoek, 2000)

2.5.4.4.2.3 Elastic Modulus Reduction Factor

The elastic modulus reduction factor is applied to the rock mass elastic modulus determined from small-strain seismic V_s measurements. The application of the reduction factor is used to estimate the elastic modulus operating at larger strains caused by static loading. The staff reviewed Deere et al. (1967) and noted that it recommended a reduction factor of 0.5 for a rock mass RQD of approximately 70 percent. The staff reviewed the RQD values for the A-series borings at the LNP Unit 2 site and questioned the justification of using a reduction factor of 0.5. In RAI 2.5.4-15, the staff asked the applicant to justify the use of a reduction factor of 0.5 in light of the Deere et al. (1967) relationship. In the same request, the staff also asked the applicant to discuss the elastic modulus values obtained from rock UCS and PMTs, since they are significantly lower than the values obtained from the V_s. The staff asked the applicant if the UCS and PMT results influenced the selection of rock mass elastic modulus values used in the design.

In its June 23, 2009, response to RAI 2.5.4-15, the applicant stated that the modulus reduction factor of Deere et al. (1967) is not applicable to the LNP site because it is an estimate based on data from high strength granite gneiss of excellent rock mass quality located within 5.4 m (18 ft) from the surface.

The applicant also explained that the depth of the PMTs was limited by the instability of open holes and was performed in only one borehole per unit. Accordingly, the applicant excluded the PMT results from the development of the elastic modulus values. The applicant also judged the Hoek-Brown factors recommended to reduce the elastic modulus based on UCS tests results to be overly conservative. The applicant also noted that since the V_s values take the site variability into account more so than the other methods, the elastic modulus values derived from the seismic measurements are the most complete account of the site variability. Therefore, the applicant concluded that the elastic modulus values derived from V_s measurements are the most representative because these values measured the highest achievable stiffness for the rock mass, including discontinuities, and a reduction factor of 50 percent accounted for the degradation of the elastic modulus due to range of deformation likely to occur at the site.

The staff reviewed the UCS test and PMT results and compared those values with the V_s-derived elastic moduli. The staff noted that there are insufficient PMT results to enable the applicant to assign material stiffness to the layers of the Avon Park Formation due to problems with keeping the borehole open during testing. Thus, the staff concludes that the PMT results could not be used for analysis purposes. The applicant noted, and the staff concurs, that similar problems limit the usefulness of the UCS test results. The staff also notes that the elastic moduli computed from the available UCS test results are typically 10 to 40 percent of the stiffness determined from the V_s results, indicating that the sampling process had a deleterious effect on the testable samples and testing unconfined samples is not representative of the in-situ stress regime.

The staff, therefore, concludes that the UCS-derived elastic modulus values were affected by sampling disturbance and unconfined testing of the samples, and concurs with the applicant that the results are not representative of the in-situ stiffness of the Avon Park limestone. The staff also concludes that the elastic moduli from the suspension P-S velocity logging surveys are the best available data to use in the engineering analyses because these data provide the most complete description of the variability at the site, represent the highest achievable stiffness measured at very small strains, and include the natural discontinuities at the in-situ effective stresses. Because the V_s were obtained in-situ at intervals of 0.5 m (1.6 ft) for the full depth of the boring, the staff notes that it provides a nearly continuous record of the stiffness of the rock mass. Furthermore, because a different rock type was used to develop the relationship proposed by Deere et al. (1967), the staff concurs that the relationship proposed by Deere et al. (1967) is not applicable to the LNP site. The staff also independently reviewed the recommendations of Mayne et al. (2002), and concludes that a reduction factor of 50 percent is adequate since it is based on a FS of 3 and is within strain levels appropriate for deformation analyses. Accordingly, the staff considers RAI 2.5.4-15 resolved.

2.5.4.4.2.4 Conclusion for Properties of Subsurface Materials

The staff reviewed the subsurface material properties, the methods used to determine those properties, and the input parameters used to estimate rock mass shear strength parameters and stiffness properties that were used as inputs in the engineering analyses. The staff observed that the applicant was conservative in its selection of the GSI, m_i, and elastic modulus reduction factor in the determination of the rock mass strength parameters. The staff therefore concludes that the use of these results in the Hoek-Brown criteria resulted in conservative rock mass strength parameters.

Based on the near continuous measurements of V_s, the staff concludes that the V_s results are the most complete picture of the in-situ conditions. Since the applicant measured the V_s profiles using the suspension P-S velocity logging methods and downhole seismic methods at LNP Units 1 and 2, and the results were consistent, the staff concludes that this proves the reliability of the V_s data. The staff concludes that the V_s data accurately characterizes the velocity profile at the LNP site, which in turn confirms the static and dynamic stiffness of the foundation materials, since those properties are derived from the V_s measurements. The use of the measured V_s and V_p to produce the maximum shear modulus and maximum elastic modulus required the applicant to apply a factor of 0.5 to reduce the elastic modulus to a value consistent with the strain level that will exist under the applied loading.

Based on its review of Mayne et al. (2002), the staff confirms that this reduction factor was supportable. Accordingly, the staff concludes that the applicant applied adequate conservatism in its selection of the material properties based on ample borings, proper sample preparation, adequate numbers of tests, redundant testing, and conservative assessments of geologic conditions at LNP Units 1 and 2. The staff further concludes that the applicant adequately addressed COL Information Item 2.5-6 and that the field and laboratory data are sufficient to determine the subsurface properties and foundation conditions in accordance with RG 1.132, Revision 2; and RG 1.138, Revision 2, and meet the criteria of 10 CFR Part 50, Appendix A, GDC-2, and Appendix S; and 10 CFR 100.23.

2.5.4.4.3 Foundation Interfaces

The staff focused its review of LNP COL FSAR Section 2.5.4.3 on the applicant's description of the topographic layout, diaphragm wall, removal and replacement of the subsurface materials down to an El. of -7.3 m (- 24 ft), RCC bridging mat, remedial grouting, and structure locations with respect to the foundation materials supporting the LNP Units 1 and 2 safety- and nonsafety-related structures.

The staff noted that many of the core runs failed to fully recover the rock core, and poor rock core recovery was a persistent occurrence across the LNP site. Due to insufficient recovery of samples of the foundation layers, the staff questioned the nature and lateral extent of the materials in the no recovery zones. Although the applicant relied on the V_s results from the suspension P-S velocity logging surveys to characterize these materials, the staff needed more information to determine if the V_s measured in the no recovery zones were representative of those materials. Therefore, the staff asked a series of questions to obtain more information about the nature of the materials that were not recovered.

2.5.4.4.3.1 Offset Boring Program

To address the staff's concerns, the applicant completed an offset boring program consisting of six newly drilled boreholes in close proximity to existing borings to better characterize the zones where material was not recovered.

The staff performed a thorough review of the offset boring program report. The borings were drilled in close proximity to A-Series borings that recorded the worst recovery, drilling four borings at LNP Unit 1 and two borings at LNP Unit 2. The offset borings, drilled to depths of 62.4 to 73.1 m (205 to 240 ft) relative to the existing surface, were offset 1.5 m (5 ft) from A-Series borings. The applicant used precision drilling tools and techniques in an effort to increase the recovery and measure the strength of the materials in the former no recovery zones. The applicant also noted the drilling time, drill bit revolution speed and drill bit thrust while coring to provide additional data that could be used to characterize the materials. The applicant also employed soil sampling and testing equipment in an effort to determine the strength of the softer materials, but this effort was largely unsuccessful as it became obvious that the softer materials were not soils and therefore not subject to soil testing techniques. The applicant replaced the double tube core barrel with a triple tube core barrel to improve recovery of the badly fractured Avon Park limestone and reduced the fluid circulation pressures from up to 3,447 kPa (500 psi) in the A-series borings to 1,034 to 2,068 kPa (150 to 300 psi) in the offset borings. The applicant also reduced the core run from 1.5 m (5 ft) down to 0.76 m (2.5 ft) to reduce the likelihood that the bottom portion of the core run was being pulverized by the upper portion of the core lodged in the core barrel. The staff observed recorded rod drops in the offset borings, which indicated the potential for voids or possibly soft materials not capable of supporting the weight of the drilling tools, but these were typically in the range of 0.06 to 0.30 m (0.2 to 1.0 ft), consistent with previous data collected and presented.

The recovery rates improved from 65 to 85 percent in the offset borings at LNP Unit 1, but because the RQD values remained essentially the same, the staff concludes that rock soundness was not the cause of the greater recovery. The O-series borings demonstrates that the low recovery rates were more closely related to drilling technique in soft rock than actual voids, and that the no recovery zones recorded in previous series borings typically resulted from weathered limestone fragments being ground up and washed away by the production drilling methods employed in the pre-offset program borings.

The results of the offset boring program also confirmed that the assumption of 13 soft zones was conservative, and that the material, which was previously postulated as soft soil infill was actually variably weathered Avon Park limestone. The staff therefore concludes that the modeling of the bedding planes with an elastic modulus of 113 MPa (16.5 ksi) in the sensitivity studies is conservative.

Finally, based on the results of the offset boring program, the staff concludes that extensive soil-filled karst features do not exist at the site, and the V_s measurements are representative of the in-place materials and can be relied upon to perform the engineering analyses. The RAIs issued to address the staff's concerns and considerations leading up to the offset boring program are detailed in the following paragraphs.

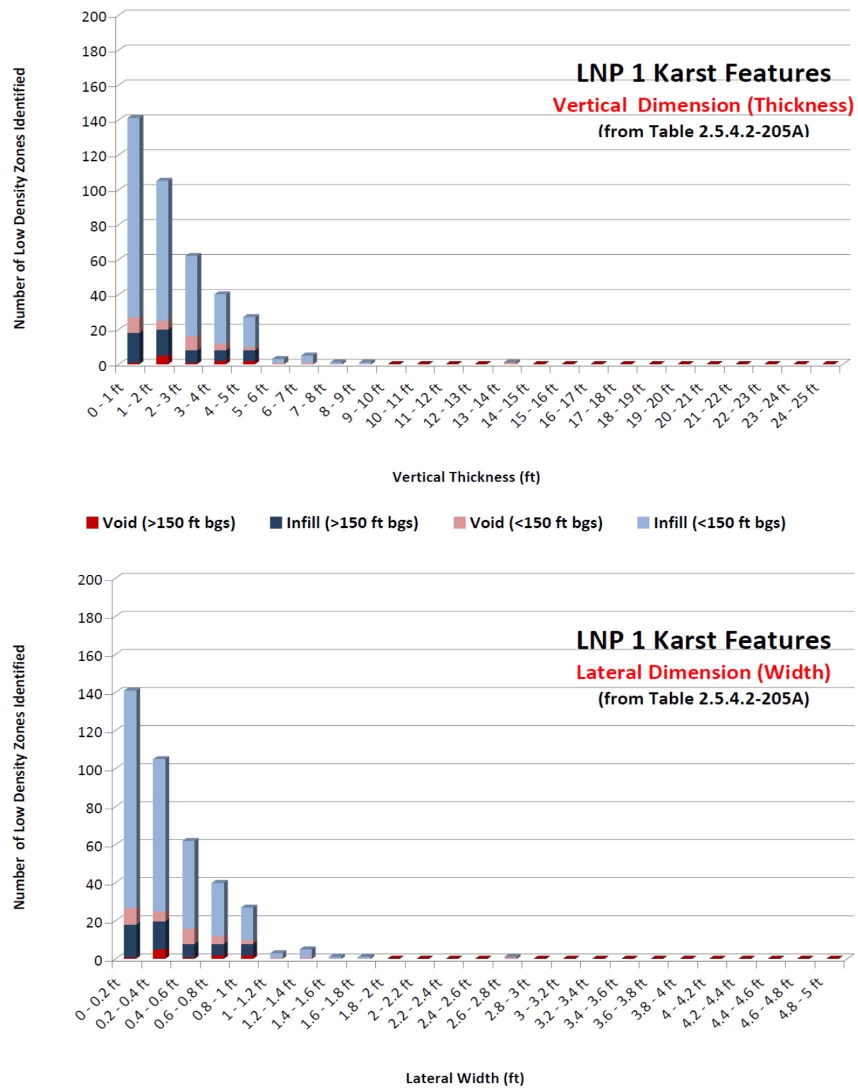
2.5.4.4.3.2 Karst Features and Voids

The staff reviewed the borings completed at the LNP site and noted that the borings revealed karst features. Accordingly, in RAI 2.5.4-1, the staff asked the applicant to justify that the boring spacing was adequate to characterize the karst features at depth, and support the conclusion of no connectivity of voids between boreholes.

In its November 20, 2008, response to RAI 2.5.4-1, the applicant stated that the potential for karst features at depth is reduced due to the nature of the karst features and the resistance of the Avon Park Formation to undergo further dissolution. The applicant characterized the karst as erosional features having a "plus-sign" morphology created by dissolution of the limestone along near-vertical fractures and at the junctures with horizontal bedding planes as the fractures dissolved. The applicant stated that the potential for ground water to dissolve limestone decreases with depth due to the reduction in the acidity of the ground water as it seeps to greater depths. Also, the applicant noted that the Avon Park Formation is highly dolomitized making it more resistant to dissolution because the dolomitic crystalline makeup inhibits the rate of karst formation.

The staff reviewed the individual borings, the geologic descriptions and driller's notes provided on the boring logs, the seismic and non-seismic geologic data, and the LNP COL FSAR tables that list the incidences of voids and soft zones encountered at the LNP Units 1 and 2 sites, respectively. The staff also reviewed the procedures the applicant used to determine the vertical and lateral dimensions of the karst features listed in the aforementioned tables and the histograms of void and soil-filled void occurrence presented in SER Figures 2.5.4-10 and 2.5.4-11 for LNP Units 1 and 2, respectively.

LNP COL 2.5-1, LNP COL 2.5-5



Progress Energy Florida
Levy Nuclear Plant
Units 1 and 2
Part 2, Final Safety Analysis Report

LNP 1 Karst Features
Vertical / Lateral Dimension
FIGURE 2.5.4.1-201A

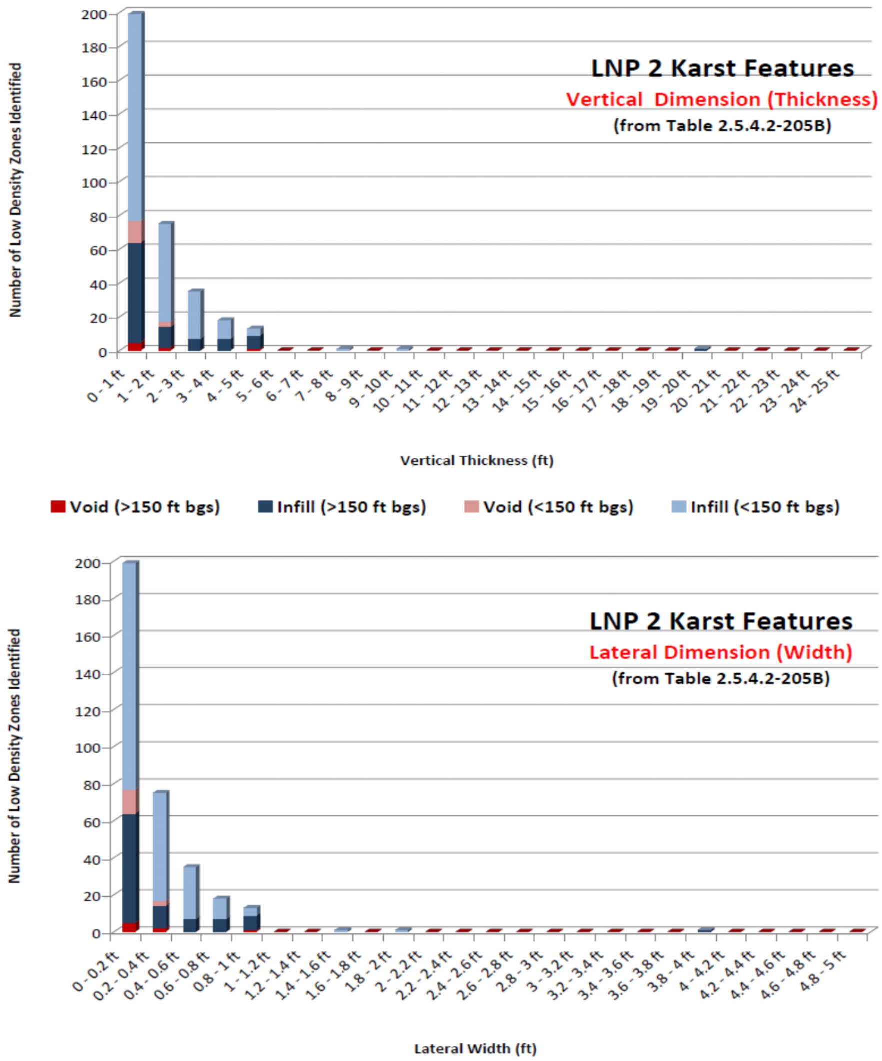
Rev 0

Figure 2.5.4-10. Distribution of Vertical and Lateral Dimension of Voids at LNP Unit 1 Below Ground Surface (bgs) (FSAR Figure 2.5.4.1-201A)

The staff finds the method the applicant used to estimate void size acceptable. The applicant calculated the theoretical volume of the borehole and subtracted that volume from the total grout take consumed in backfilling the borehole. Depending on whether the applicant determined the void was vertically or horizontally oriented, the applicant increased the excess volume by either 50 percent for the vertically oriented voids or 100 percent for the laterally oriented voids. The total volume of excess grout volume was then applied to a specific void located in the borehole to calculate the void dimensions. The staff concludes that this approach resulted in conservative estimates of the void dimensions, because the applicant increased the volume of grout take by 50 and 100 percent as explained above.

SER Figures 2.5.4-10 and 2.5.4-11 show the distribution of voids and soil-filled voids observed in the borings above and below 45 m (150 ft) below the ground surface. These figures illustrate that the majority of the karst features are infilled. The applicant determined from the offset boring programs that what was initially postulated as infilled voids is now recognized as being severely weather Avon Park limestone, hence the frequency of postulated voids would be dramatically reduced in this figure. Some actual voids noted by rod drops were observed in the exploration borings preceding the drilling of the offset borings and are accounted for in SER Figures 2.5.4-10 and 2.5.4-11. The staff also observed that the frequency of occurrence of voids is greatest at LNP Unit 1 and typically occurs above a depth of 45 m (150 ft). The offset boring program, which was drilled with greater precision, also had some rod drops. The staff noted that these rod drops could either represent actual voids or very soft soils, but whatever the case, the vertical drops were small, typically less than 0.3 m (1 ft) in height. Based on SER Figure 2.5.4-11, the staff further observed that the largest postulated soil filled void has a vertical dimension of 6 m (19.5 ft). This karst feature was encountered at LNP Unit 2 in boring A-11, which is within the footprint of the nuclear island between the depths of 70.4 and 76.3 m (231 and 250.5 ft). The A-11 boring log does not indicate rod drops, and notes that the drilling time throughout this interval was 2 to 3 minutes. Since there was no recovery, the applicant included it as a postulated soil-filled void, but with the better understanding obtained from the offset boring program, the applicant stated the more likely explanation is that this zone is weathered, soft Avon Park limestone.

LNP COL 2.5-1, LNP COL 2.5-5



Progress Energy Florida
Levy Nuclear Plant
Units 1 and 2
Part 2, Final Safety Analysis Report

LNP 2 Karst Features
Vertical / Lateral Dimension
FIGURE 2.5.4.1-201B
Rev 0

Figure 2.5.4-11. Distribution of Vertical and Lateral Dimension of Voids at LNP Unit 2 Below Ground Surface (bgs) (FSAR Figure 2.5.4.1-201B)

Given that karst in the region commonly developed in association with the "plus-sign" morphology, in which most dissolution occurs along vertical joints and at intersections of the joints with bedding planes, the staff concludes that much of the void development is limited to vertical joints and junctures between the vertical joints and bedding planes. Many borings deeper than 61 m (200 ft) intersected horizontal bedding planes without yielding evidence of extensive voids, leading the staff to conclude that maximum lateral void dimensions were conservatively estimated. Also, because data shown in SER Figures 2.5.4-10 and 2.5.4-11 illustrate that karst features are predominately located within a depth of 45 m (150 ft) below the existing ground surface, the staff concludes that certain parts of the applicant's remedial ground improvement plan could potentially help to minimize concerns about extensive voids in materials underlying safety-related structures. For example, the depth range that includes most of the karst features will be grouted in the interval from 22.8 to 45 m (75 to 150 ft) and the grouted zone excavated from the ground surface down to a depth of 22.8 m (75 ft), effectively minimizing the risk of collapse due to the presence of karst features beneath the nuclear island. However, the staff recognizes that no part of the secondary, primary, or tertiary grouting programs is intended or required by the applicant to perform a safety function.

Due to the small dimensions of actual voids, the staff concludes that borehole spacing is sufficient and further assessment of the connectivity of dissolution features between boreholes is not necessary. Furthermore, given the applicant's remedial ground improvement plan combined with the reduced ability for further dissolution due to dolomitization, as well as the lack of impact of voids at depth on safety-related structures, the staff concludes that the characterization of karst features is adequate. Based on the details of the drilling program in response to RAI 2.5.4-5 and the staff's conclusion that the material in the no-recovery zones is weathered-in-place Avon Park limestone, RAI 2.5.4-1 is resolved.

2.5.4.4.3.3 Uniformity Criteria Adherence

The staff reviewed the uniformity criteria outlined in the AP1000 DCD described below and speculated that, due to the presence of karst features and highly variable RQDs, the LNP site may be non-uniform. In RAI 2.5.4-2d, the staff asked the applicant to provide a detailed explanation of how the limestone supporting the RCC bridging mat meets the uniformity requirements for subgrade reaction described in the AP1000 DCD.

The applicant stated that, consistent with the AP1000 DCD, the Avon Park limestone meets the uniformity requirements for thickness, dip and variation in V_s down to the depth of interest at 36.5 m (120 ft) below grade. The applicant noted that "beneath the RCC bridging mat, one geologic unit is uniformly present to depths beyond [47.5 m] 150 feet below grade, consistently across all boreholes within the nuclear island footprint, meeting the thickness requirement of a uniform site." The applicant also noted that the dip angle is approximately 2 degrees for both LNP Units 1 and 2, which is within the 20 degree requirement for a uniform site given in the AP1000 DCD. Finally, the applicant noted that smooth variations in the average V_s exist between borings within the Avon Park limestone layers, but the averages between borings are within the 20 percent variation allowed by the AP1000 DCD. Based on the uniformity criteria of the AP1000 DCD, the applicant concluded that the LNP site was uniform.

The staff reviewed the boring logs presented in LNP COL FSAR Appendix BB, the results of downhole and suspension P-S velocity logging surveys, and the dip of the limestone layers beneath LNP Units 1 and 2. The staff confirmed that the thicknesses of the individual layers were uniform, and that the maximum dip of any layer was on the order of 2 degrees. The staff also noted that the average V_s in any boring was within 20 percent of the average of all the borings within a given layer and this uniformity exists to at least 36.5 m (120 ft) below grade. The staff compared these results to the AP1000 DCD and concludes that the site meets the uniformity criteria set forth in the DCD. Accordingly, RAI 2.5.4-2d is resolved.

2.5.4.4.3.4 Drilling Methods

The staff reviewed the subsurface exploration plan, including the applicant's extraction of 1.5 m (5 ft) long rock cores at various depths in the subsurface in order to obtain the RQD and recovery data. The staff noted that although the drilling time was recorded, the logs did not record the thrust or rotational speed of the drill bit, which would assist in characterizing the materials not recovered. In RAI 2.5.4-5, the staff asked the applicant to provide the drilling pressures that coincide with the time of core drilling, to aid the staff in its effort to determine if the no-recovery zones were voids, soil-filled karst features, or unrecoverable weathered limestone.

In its April 2, 2009, response to RAI 2.5.4-5, the applicant stated that, because it is not normal engineering practice, it did not record the drilling pressures or the drill bit revolutions per minute. The applicant also stated that suspension P-S velocity logging in the I-series boreholes was fair to poor or undecipherable because of the sonic drilling technique and poor coupling of the casing with the borehole sidewall, but noted that the results of other geophysical surveys yielded useful data. The applicant provided the additional caliper, acoustic televiewer, and downhole geophysical data used in the engineering analysis to define karst features in the subsurface. For the Avon Park Formation, the applicant used the mass properties in the engineering analyses, and assumed all karst features were voids. This removed the need to define the engineering properties of in-fill materials. Finally, the applicant described plans to obtain the strength and consolidation properties of in-filled and/or weathered-in-place materials as part of the offset boring program.

During the drilling of the offset borings, the applicant recorded drill pressures, rotational drill speed, time of drilling, as well as other data, and attempted to obtain samples for laboratory testing. The applicant compared the results of the offset boring program to those used in the geotechnical analyses performed at the LNP site and concluded that the engineering properties were conservative.

On January 19, 2010, the applicant supplemented its initial response to RAI 2.5.4-5 to include a description of and the results obtained from the offset boring program. The staff's review of the offset boring program is discussed above. Because the applicant recorded the time of drilling, drill bit rotational speed and drill pressures, the staff confirmed that the no recovery zones were not voids, nor contained soft infilled soils, but were characterized as variably weathered Avon Park limestone. Thus, RAI 2.5.4-5 is resolved.

2.5.4.4.3.5 Karst Feature and Void Dimensions Based on Grout Takes

The applicant had estimated the size of actual voids from grout takes measured while backfilling selected core borings made during its exploratory program. The staff reviewed the methodology the applicant employed in determining void size, which consisted of comparing the total grout take to the theoretical volume needed to backfill the boring. The applicant conservatively increased the excess grout volume by 50 or 100 percent depending on the orientation of the void under consideration, and used that volume to estimate the void dimensions. The staff determined that additional information was needed to ensure that this methodology was conservative as the staff postulated that some void volumes could be underestimated if voids contained soil infill, which would effectively reduce the amount of grout take. RAI 2.5.4-6 asks the applicant to confirm that void volumes measured by grout takes were representative of the dimensions of karst features.

In its April 2, 2009, response to RAI 2.5.4-6, the applicant referred to the LNP COL application supplemental information dated September 12, 2008, and the responses to RAIs 2.5.1-5 through 2.5.1-7, 2.5.4-1 and 2.5.4-3a, which describe how the grout take was used to estimate the lateral extent of karst features. The applicant also referred to the response to RAI 2.5.4-8 for the results of additional analyses that modeled “bedding planes” of infilled or weathered-in-place materials instead of voids.

On January 19, 2010, the applicant supplemented its response to RAI 2.5.4-6 to include the results of the offset boring program. The applicant identified the low recovery zones as severely weathered or degraded dolomite that was weathered-in-place and not infilled material as previously identified. Based on the results obtained, the applicant concluded that the grout data analyses to determine the extent of the possible karst features were adequately conservative. The results of the offset boring program indicate that what was once considered soil-filled karst features are actually weathered limestone zones; therefore, the applicant concluded that the size of postulated voids of 3 m (10 ft) in diameter is conservative. Finally, the applicant concluded that the use of soil properties for the material assumed to exist continuously along bedding planes is conservative. The applicant has revised the FSAR to incorporate this additional information.

The staff reviewed the LNP COL FSAR, the related supplemental materials, and the responses to the cited RAIs and concludes that the materials left undocumented in the “no recovery” zones in borings performed during previous explorations were not soil-filled voids as was initially postulated. The staff also reviewed the offset boring program and confirmed that the “no recovery” zones typically contained highly fractured, severely weathered in place materials from the Avon Park limestone parent rock. The staff also concludes that the applicant’s estimate of the size of the voids to be no larger than 3 m (10 ft) in diameter, as determined by grout takes, was sufficiently conservative and supported by the results of the offset boring program.

Finally, the staff reviewed the V_s results and concludes that the V_s measured in the weathered zones (no recovery zones) are similar to other zones of the Avon Park limestone where recovery was made. The staff concludes that had large voids been present in the rock profile, it would have been reflected by the V_s due to the 0.5 meter sampling interval. SER

Figure 2.5.4-12 shows the V_s measured in borings along the N-S profile at LNP Unit 1. The blue dots represent individual V_s measurements and the blue line is the average of those measurements. Adjacent to the V_s profiles are plotted the sample recovery percentages and the RQDs determined during drilling that correspond to the V_s profiles. The lighter red line in this figure represents the sample recovery and the solid green line the RQD. The staff observed no consistent correlation between low recovery and V_s. From this figure, it is apparent to the staff that even at very low recovery rates, the V_s is typically greater than 457 m/s (1,500 fps). From the V_s data the staff concludes that the low recovery zones do not represent voids or soil-filled voids, which was later confirmed by the offset boring program. The staff, therefore, concludes that the maximum size of a void of 3 m (10 ft) diameter is conservative. Accordingly, RAI 2.5.4-6 is resolved.

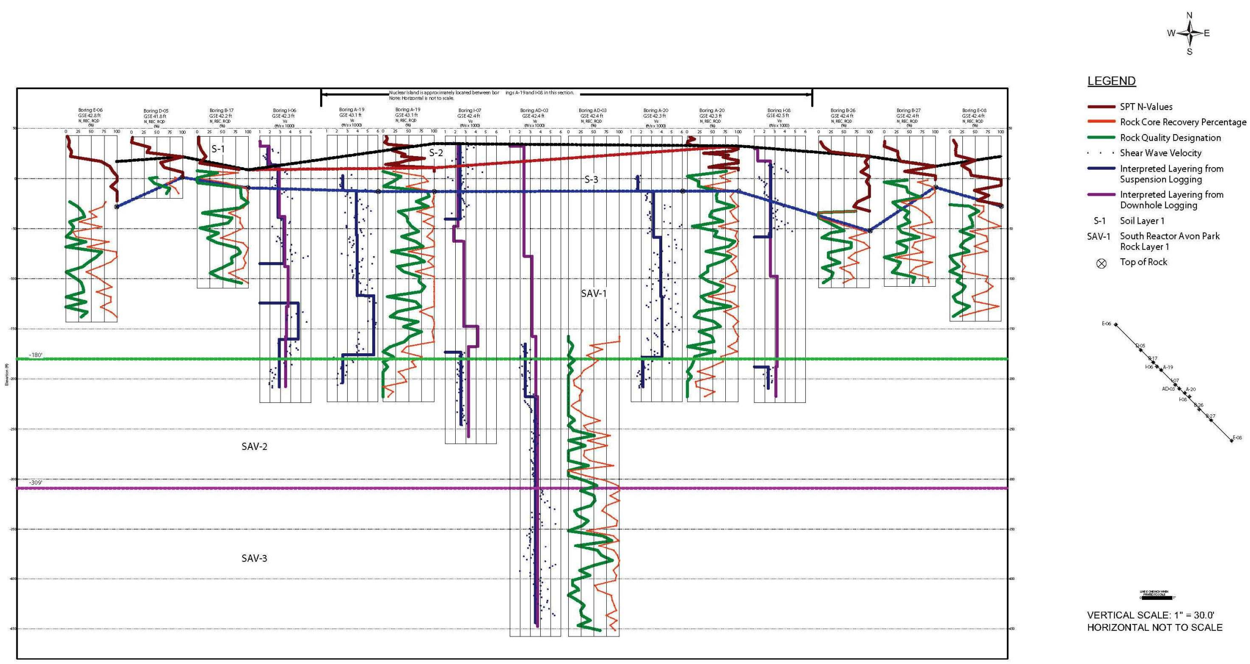


Figure 2.5.4-12. V_s Profiles in Boring AD-03 and Boring AD-20

- (Excerpted from FSAR Figure 2.5.4.2-204B)

2.5.4.4.3.6 Low Recovery of Core Samples

To the staff, it appeared that the recovery rates of rock core samples varied across the site and with depth. Considering that the low recovery rates could be indicative of softer rock, the staff asked the applicant in RAI 2.5.4-9 to describe what considerations it gave to the spatial variation of the low recovery rates.

In its April 2, 2009, response to RAI 2.5.4-9, the applicant stated that the higher recurrence of lower recovery in the center borings results from a vertical variation and noted that adjacent boreholes show similar lower recoveries at the same elevations. The applicant provided a figure illustrating the vertical distribution of the recovery in the boreholes beneath the nuclear island and stated it accounted for this vertical variability in the sensitivity analyses. The staff further noted that the sensitivity analyses conservatively consider the presence of continuous infilled and/or weathered-in-place material along the bedding planes, which accounts for much of the low or no recovery zone material.

The staff reviewed the applicant's response and the boring data. Based on the results of the borings, especially the additional O-series borings drilled in close proximity to the A-series borings, it was apparent to the staff that the high rate of no recovery or poor recovery was a product of the drilling equipment and practices the applicant used during the initial site exploration program. Based on the offset boring results, the staff concludes the recovery rates from the initial boring programs are not useful for identifying soft zones or variations in rock stiffness. The staff therefore relied on the results of the V_s profiles to gauge uniformity of mass rock stiffness, which proved to have uniform average velocity profiles, as observed in SER Figure 2.5.4-12. The staff further concludes that the sensitivity analyses the applicant performed adequately consider any spatial variation. Those analyses are considered in Section 2.5.4.4.10. Accordingly, RAI 2.5.4-9 is resolved.

2.5.4.4.3.7 Grouting of Karst Features

In LNP COL FSAR Section 2.5.4.7, the applicant stated that the purpose of the grouting program was to create a semi-impermeable barrier to reduce ground water inflow into the excavation thereby reducing dewatering requirements. In RAI 2.5.4-17, the staff asked the applicant to clarify this statement that all karst features will be eliminated by the grouting program, discuss any plans for additional exploration that will be implemented to identify karst features to target during the grouting phase, and describe how it will assess whether all the karst features have been eliminated.

In its June 9, 2009, response to RAI 2.5.4-17, the applicant stated that it did not plan any additional site explorations to identify karst features. The applicant also clarified that the statement in question refers to the elimination of known karst features, revised the FSAR to remove the statement in question and referred to FSAR Section 2.5.4.12 for additional details of the subsurface improvements at the site.

The staff reviewed the RAI response, including the FSAR revisions, the completed grout test program, the proposed grouting plan and the referenced FSAR Section 2.5.4.12, and concludes that the applicant proposed satisfactory engineering solutions to grout the eroded vertical joint sets and bedding planes. The staff also concludes that the proposed use of grout holes, including inclined grout holes if deemed necessary, spaced on 4.8 m (16 ft) centers as primary grout points, followed by split-spaced grout holes on 2.4 m (8 ft) centers to an El. of -30.1 m (-99 ft), is an acceptable approach to cutoff seepage. The staff notes that the combination of inclined and split spaced grout holes has a large probability of filling the stipulated vertical joint sets and bedding planes in the Avon Park Formation. The staff also finds that the applicant's commitment to perform a tertiary stage of grouting on 1.2 m (4 ft) centers during excavation activities if the first and second stage grouting does not achieve the desired seepage cutoff is acceptable. The staff also notes that the foundation system is designed to accommodate isolated voids up to 3 m (10 ft) in size, which is at least double the conservatively estimated lateral dimension of any actual void intercepted. Finally, the staff acknowledges that the grout program is not intended to strengthen the foundation, but only reduce inflow into the excavation. Filling of all the voids is therefore not required for stability.

The staff concludes that the proposed grouting plans will minimize seepage into the excavation, reduce pumping requirements, and stabilize the excavation bottom against uplift. The staff further concludes that the combination of the diaphragm wall, grouting program and RCC bridging mat will improve the foundation conditions without the need to fill every joint or open bedding plane. Thus, RAI 2.5.4-17 is resolved.

2.5.4.4.3.8 Conclusion for Foundation Interfaces

Based on the information and findings provided in LNP COL FSAR Section 2.5.4.3, as well as the results of the offset boring program, the staff concludes that the applicant implemented significant and adequate subsurface investigations in relation to the AP1000 safety-related structures at the LNP site to resolve COL Information Item 2.5-5 and COL Information Item 2.5-6 related to foundation interfaces. The staff further concludes that the applicant adequately investigated the subsurface materials beneath the nuclear island construction zone for LNP Units 1 and 2 and beneath the surrounding and adjacent structures. The staff based its conclusions on: (1) its review of plot plans showing the locations of all site explorations, such as borings, seismic and non-seismic geophysical explorations, piezometers, geologic profiles, and the locations of the safety-related facilities; (2) its review of the profiles the applicant presented, illustrating the detailed relationship of the foundations of all seismic Category I and other safety-related facilities to the subsurface materials; and (3) its review of core borings, SPT borings, V_s profiles and non-seismic geophysical logging results. Accordingly, the staff concludes that the foundation interfaces as described in FSAR Section 2.5.4.3 form an adequate basis for the characterization of the foundation interfaces at the LNP site and meets the requirements of 10 CFR Part 50, Appendix A, GDC 2 and Appendix S; and 10 CFR 100.23.

2.5.4.4.4 Geophysical Surveys

The staff focused its review of FSAR Section 2.5.4.4 on the adequacy of the applicant's geophysical investigations to determine soil and rock dynamic properties. The applicant performed both seismic and non-seismic geophysical surveys to characterize the subsurface geology beneath the LNP site. The applicant relied primarily on the suspension P-S velocity logging method to determine the site stratigraphy and provide the engineering properties of subsurface materials, particularly from V_s and V_p profiles. As a secondary method, the applicant performed downhole V_s surveys to confirm the results obtained from the suspension P-S velocity logging. In addition, the staff considered the acoustic televiewer surveys for information regarding verticality of the borehole including graphic images to examine joints and fractures and calculate dip and orientation of planar fractures. Non-seismic surveys of the boreholes included natural gamma, gamma-gamma (density), neutron-neutron (porosity) and induction (conductivity) surveys. The staff also referred to the results of the non-seismic tests for information on the lithology and stratigraphy, location of low density zones, presence of clay, and variations in moisture content.

Based on the results of the suspension P-S velocity logging surveys, the applicant developed the engineering properties for the various layers of the Avon Park limestone. The staff considered the possibility that the suspension P-S velocity logging surveys averaged the velocities of softer zones or voids with denser zones that might occur over the measurement interval. In RAI 2.5.4-4, the staff asked the applicant to discuss the possibility that near-horizontally oriented lenses of soft material were missed or averaged with the high velocities of the adjacent rock. The staff also asked the applicant to describe how it accounted for the variability of the suspension P-S velocity logger results.

In its April 2, 2009, response to RAI 2.5.4-4, the applicant stated that although there was a 1-m (3.2-ft) separation between the receivers used in the suspension logging probe, it measured at 0.5 m (1.6 ft) increments to ensure that the receivers would not completely miss any near-horizontally oriented lenses of soft material. The applicant noted that this interval would also reduce the effect of averaging that is apparent in larger increments. The applicant stated that although the analysis ignored the structural capability of infilled or weathered-in-place materials, these materials were considered in the development of the mass strength and stiffness properties. The applicant referred to the sensitivity analyses provided in response to RAIs 2.5.1-7 and 2.5.4-2, which show that these features are acceptable as voids without any structural capacity. Finally, the applicant performed a sensitivity analysis to address the potential variability of the subsurface materials, and stated that the properties assigned to the postulated continuous bedding features were shown to be less than the properties of the materials revealed by the offset boring program investigation described in SER Section 2.5.4.4.3.

Based on the response to RAI 2.5.4-4, the staff concludes that the smaller measurement interval of 0.5 m (1.6 ft) reduces the possibility that the near-horizontally oriented layers of highly weathered Avon Park limestone or soil in-filled zones would be completely missed and minimizes the effect of averaging softer layers with harder layers. The staff also observed that the variability in the measured velocities throughout the depth of the rock profile is a good indication that the suspension P-S velocity logger detected layers of softer materials interbedded with harder limestone. The staff further notes that because the offset boring program found that the interbedded materials are typically severely degraded weathered-in-place Avon Park limestone, as opposed to soil in-fill, the suspension P-S velocity logging results are representative of the V_p and V_s of individual layers within the Avon Park limestone. Accordingly, RAI 2.5.4-4 is resolved.

The staff also considered the results of non-seismic natural gamma, gamma-gamma, neutron-neutron and induction surveys and concluded that the results suggested continuous low density zones of large areal extent do not exist below the founding level of the RCC bridging mat. The staff further notes that in comparing low density zones to available V_S profiles at similar elevations, the V_s profiles do not fall below 457 m/s (1,500 fps), which is above the 305 m/s (1,000 fps) required by the AP1000 DCD. The staff also observed that the localized low density zones typically fall above the base of the RCC bridging mat, or within the zone to be grouted, and therefore will either be removed and replaced or improved where they do occur below the base of the RCC bridging mat.

The NRC staff reviewed the results of the geophysical surveys, specifically the profiles of V_s and V_p, RQD, percent recoveries, and SPT N-values presented on the geophysical cross-sections in LNP COL FSAR Section 2.5.4.4, the results of non-seismic geophysical surveys presented in response to RAI 2.5.4-5, the applicant's response to RAI 2.5.4-4, and boring logs presented in Appendix BB of the LNP COL FSAR to ensure that the applicant obtained sufficient data to ascertain the soundness and integrity of the rock mass and derived the static and dynamic engineering properties for use in engineering analyses. Based on the applicant's site investigation program and results, the staff concludes that the applicant performed a complete and thorough geophysical survey of the LNP site using a variety of geophysical testing methods. Accordingly, the staff concludes that the applicant adequately addressed COL Information Item 2.5-6. The staff also concludes that the V_s described in FSAR Section 2.5.4.4 addresses Interface Item 2.12. The staff further concludes that the geophysical tests and methods described in FSAR Section 2.5.4.4 form an adequate basis for the geophysical surveys of the LNP site and meets the requirements of 10 CFR 100.23.

2.5.4.4.5 Excavation and Backfill

The NRC staff focused its review of FSAR Section 2.5.4.5 on the horizontal and vertical extent of all seismic Category I excavations, fills, and slopes, ground water conditions and geologic features, the backfill sources, types and quantities of backfill, static and dynamic engineering properties of backfill, compaction specifications, and soil retention system. The staff also considered the applicant's description of the sequence of excavation and backfill plans, particularly the placement of grout between an El. of -7.3 and -30.1 m (-24 and -99 ft), the installation of the diaphragm walls, and the method of excavation and subgrade preparation. The staff noted the applicant's intent to remove and replace the subsurface materials down to an El. of -7.3 m (-24 ft) from which it will construct the 10.7 m (35 ft) thick RCC bridging mat. The applicant stated that backfill between the diaphragm wall and the nuclear island will consist of a low strength concrete-type backfill placed up to the top of the diaphragm wall at approximately an El. of 12.8 m (42 ft). Backfill added above the existing site topography to final site grade at an El. of 15.5 m (51 ft) will be an engineered backfill. The RCC bridging mat is a structural element and is reviewed and discussed in SER Sections 3.7 and 3.8.

2.5.4.4.5.1 Backfill Adjacent to the Nuclear Island

The staff reviewed the use of low strength concrete-type backfill, specifically the CLSMs as backfill material adjacent to the sidewalls of the nuclear island. The staff is familiar with the use of the CLSM for backfilling utility trenches. The use of CLSM has advantages over soil backfill. For example, it typically has strength greater than 3,450 kPa (500 psi) and is easier to place in confined spaces than conventional soil backfill. However, the staff needed additional clarification regarding the potential for long-term strength loss in CLSM due to the leaching out of cementatious bonding materials.

In RAI 2.5.4-22, the staff asked the applicant to justify use of CLSM and address the issue of long-term stability, and provide the design standards, as well as to discuss the construction quality control plans to ensure uniform placement of the CLSM.

In its June 23, 2009, response to RAI 2.5.4-22, the applicant referred to the response to RAI 2.5.4-19 for a discussion of the sliding stability of the nuclear islands under seismic loading conditions. The results of these analyses indicate that the low-strength concrete-type backfill requires no shear capacity and is not subject to long-term stability concerns. The staff reviewed the advantages of using the CLSM the applicant outlined, including ACI 229R-99, "Controlled Low Strength Materials," from which the applicant cited the typical engineering properties and quality control program. The staff notes that the applicant applied the same design standards used for volumetric backfill to the CLSM but this was not reflected in the FSAR. Accordingly, the applicant updated the FSAR to refer to ACI 229R-99.

The applicant revised its response to RAI 2.5.4-22, in a letter dated September 3, 2009, to refer to the revised response to RAI 2.5.4-19, which states that there is no requirement for passive resistance provided by the backfill material adjacent to the nuclear island to remain stable against sliding or overturning. The applicant concluded that there are no concerns about long-term stability of the CLSM because there is no shear capacity requirement for the CLSM.

The staff noted that there is no requirement for passive resistance to achieve sliding stability. This issue was resolved as part of AP1000 RAI TR85-SEB1-10R4 addressing sliding stability. NUREG-1793 and its supplements document the NRC staff's review of the sliding stability analyses performed by Westinghouse for a variety of soil and rock conditions. In NUREG-1793, the staff noted that no backfill passive soil resistance was considered in the analyses and that the AP1000 DCD applicant modeled a lower frictional resistance of 0.55 consistent with the waterproof barrier. The AP1000 DCD applicant performed the analyses using the SSE free-field peak ground acceleration of 0.30g with modified RG 1.60 response spectra, and determined that the displacements were negligible. As documented in Section 3.8.5 of NUREG-1793 and its supplements, the staff accepted the Westinghouse analyses and concluded that passive resistance is not required for sliding stability.

The staff finds that the seismic demand at LNP is significantly less than that used in the analysis for the AP1000 DCD indicating that the dynamic response would be proportionately smaller than that determined in the generic AP1000 design. Since there is no passive resistance requirement at the higher ground motion, the staff concludes CLSM does not have a strength requirement. The staff, therefore, concludes that the CLSM as backfill along the sidewalls of the nuclear island is acceptable. This resolves RAI 2.5.4-22.

2.5.4.4.5.2 Engineered Backfill

The staff noted that the LNP COL FSAR provides limited information regarding the engineered backfill to bring the site to plant grade at an El. of 15.5 m (51 ft). In RAI 2.5.4-26, which was issued in response to RAI 2.5.4-24, the staff asked the applicant for details regarding the source, quantity, compaction specifications and soil properties of the engineered backfill. The applicant was also asked to justify the assumed V_s of 304.8 m/s (1,000 fps) for the backfill used to determine the peak ground acceleration,

The applicant responded to RAI 2.5.4-26 specifying the properties of the engineered fill being placed to bring the site to plant grade. The applicant stated that it did not formally establish the source of the backfill. The applicant places the total volume of engineered fill in the range of 764 to 1,529 cubic meters (1,000 to 2,000 cubic yards) placed within the limits of the diaphragm wall. The applicant stated that the backfill will be a sand fill with variable amounts of silt and clay classified by the Unified Soil Classification System as well-graded sand (SW), silty sand (SM) or clayey sand (SC), compacted to 95 percent of the relative compaction in accordance with ASTM D-1557 (2009) at plus or minus 2 percent of the optimum moisture content. The applicant assumes that the wet unit weight will be on the order of 1,762 kilograms per cubic meter (kg/m^3) (110 pcf), with a V_s in the range of 152 to 305 m/s (500 to 1,000 fps).

In determining what value to use for the V_S, the applicant performed a dynamic sensitivity analysis, which varied the V_s of the engineered fill by values of 152, 259 and 305 m/s (500, 850 and 1,000 fps). The results of this analysis are provided in SER Figure 2.5.4-13, which compares the computed effective cyclic shear stresses between an El. of 10.9 and -41.1 m (36 and -135 ft) for variable V_s.

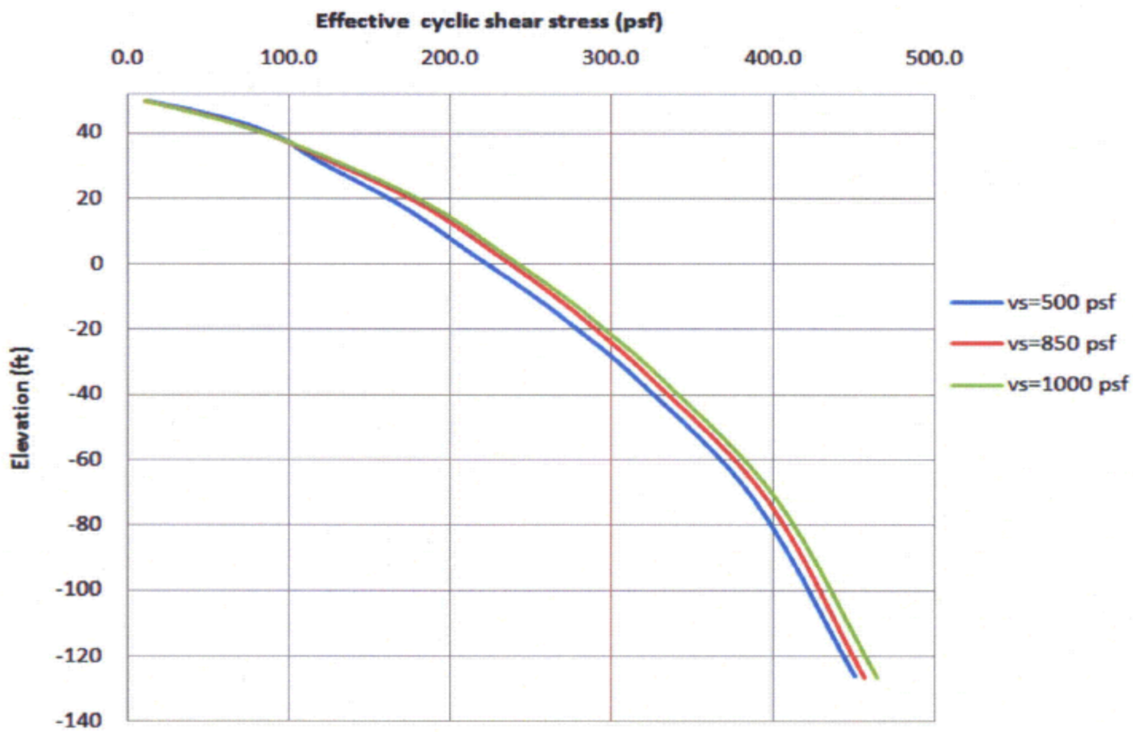


FIGURE RAI 02.05.04-26-2
LNP1 EFFECTIVE CYCLIC SHEAR STRESSES VS ELEVATION

Figure 2.5.4-13. Effective Cyclic Shear Stress as a Function Of Variable V_s Assumptions in the Engineered Fill (RAI Figure 2.5.4-26-2)

SER Figure 2.5.4-13 shows that the shear stresses generated in the underlying materials are only affected minimally by varying the V_s in the engineered fill, and that an assumption of 305 m/s (1,000 fps) results in the most conservative response, i.e., the highest generated shear stresses. The applicant concluded that this was conservative and selected 305 m/s (1,000 fps) to use in its liquefaction reanalysis.

The staff reviewed the response to RAI 2.5.4-26 and concludes that the applicant followed the guidance of RG 1.206 in providing the attributes of the engineered fill proposed for backfilling and bringing the site to final grade. The staff also concludes that the applicant adequately addressed the staff's concern regarding the use of V_s of 305 m/s (1,000 fps) for the engineered fill. The assumed values of 152, 259 and 305 m/s (500, 850 and 1,000 fps), span the range of V_s that could be expected placing a granular fill with variable fines to 95 percent relative compaction. The fact that the assumed V_s of 305 m/s (1,000 fps) results in the highest effective cyclic shear stresses addresses the staff concern that the assumption of 305 m/s (1,000 fps) was conservative. Finally, the staff concludes that the applicant satisfactorily addressed the staff's request for additional information regarding the engineered backfill; therefore, RAI 2.5.4-26 is resolved.

2.5.4.4.5.3 Conclusion for Excavation and Backfill

Based upon its review of LNP COL FSAR Section 2.5.4.5, the staff concludes that the applicant developed and described a complete excavation plan for the LNP site, including the extent of the excavations and the sequence of construction. The staff notes that the depth of the excavation extends to an El. of -7.3 m (-24 ft) with backfill to an El. of 3.3 m (11 ft) made by the placement of a 10.7 m (35 ft) thick RCC bridging mat. The staff concludes that the removal of the existing weathered Avon Park limestone and replacement with a uniform RCC is a significant improvement in the foundation conditions. Regarding the use of CLSM as backfill between the nuclear island sidewalls and the diaphragm wall, the staff concludes that this material will provide uniform backfill and fewer difficulties during placement than with attempting to place engineered fill in the space between the diaphragm wall and the nuclear island. Likewise, the applicant has not yet identified the source of the engineered fill proposed to bring the site to final grade, but the assumed properties of the engineered fill are conservative, and its potential for liquefaction is negligible. Since the engineered fill is not required for overturning or sliding stability, the staff concludes that the information provided in response to RAI 2.5.4-26 regarding the material properties of the engineered fill are sufficient to address COL Information Item 2.5-7. Accordingly, the staff concludes that the applicant adequately addressed COL Information Item 2.5-7. The staff further concludes that the excavation and backfill plans described in FSAR Section 2.5.4.5 form an adequate basis for the excavation for the nuclear islands, and the backfilling operations to bring the LNP site to grade, and meets the requirements of 10 CFR Part 50, Appendix A, GDC 2, and Appendix S.

2.5.4.4.6 Ground Water Conditions

The staff reviewed FSAR Section 2.5.4.6 where the applicant presented the ground water table conditions and construction dewatering plan. The staff reviewed the assumptions the applicant made in the design of the dewatering system and the uplift calculations. The applicant assumed a ground water table elevation of El. of 13.1 m (43 ft), which is coincident with the existing ground surface. The applicant plans to use the diaphragm wall and grouted zone between an El. of -7.3 to -10.6 m (-24 to -99 ft) to form a relatively impermeable barrier to lateral and upward seepage into the excavation. The staff noted that with these barriers in place, the applicant conservatively calculated an inflow rate of approximately 1,892 liters per minute (lpm) 500 gallons per minute (gpm). Considering this inflow rate, the applicant planned to dewater the excavation with six shallow wells using submersible sump pumps placed inside of the diaphragm wall, each with a capacity of 378 lpm (100 gpm). The applicant also planned to place sump and sump pumps at low points in the excavation to handle surface runoff. The applicant also planned for additional grouting to reduce the inflow rate if it should exceed the dewatering system capacity.

The staff noted that the applicant conducted an uplift analysis to ensure the safety of the bottom of the foundation considering the proposed dewatering scheme. In RAI 2.5.4-20, the staff asked the applicant to provide a sample calculation of the uplift analysis including figures showing the assumptions made.

In its June 9, 2009, response to RAI 2.5.4-20, the applicant presented the analyses for local piping conditions and general failure caused by uplift at the base of excavation. Piping in this context is the concentrated flow of water into the excavation caused by excess head. Regarding uplift, if the buoyant forces on the bottom of the excavation exceed the resistance offered by the weight and strength of the foundation, the foundation may heave. The applicant provided the uplift analysis for LNP Unit 2 because it is the more critical case due to the lower shear strength of the foundation limestone. In this case, the applicant assumed uplift on a block having a width equal to half of the diaphragm wall penetration depth. The applicant calculated a FS against uplift of 4.3.

The staff reviewed the applicant’s calculations and concludes that the calculated FS of 4.3 was satisfactory for the temporary condition. The staff notes that the applicant’s assumptions of unit weight and shear strength values used in the analysis were conservative, and that the calculated FS against uplift is sufficiently large to preclude a blowout of the foundation bottom. The staff also concludes that the cementatious nature of the limestone would prevent piping. Finally, the staff concludes that safety of the temporary excavation is further enhanced by the applicant’s plans for additional grouting or additional dewatering wells, if required, to control groundwater inflow and ensure a safe excavation bottom. Based on the computed FS, the conservatism in the assumptions, and the temporary nature of the excavation, the staff concludes that the foundation excavation is safe against heave and/or piping. Accordingly, RAI 2.5.4-20 is resolved.

Based upon its review of FSAR Section 2.5.4.6, the staff concludes that the applicant conservatively assumed the ground water table at the existing ground surface in the design of its dewatering system. The staff concludes that the dewatering plan is adequate to ensure the safety of the excavation. The staff further concludes that the description of the relationship between ground water, excavation, backfill, and the foundations of structures as described in FSAR Section 2.5.4.6 for the LNP site addresses COL Information Item 2.5-8, COL Information

Item 2.5-6 related to ground water conditions, and meets the requirements of 10 CFR Part 50, Appendix A, GDC2, and Appendix S; and 10 CFR 100.23.

2.5.4.4.7 Response of Soil and Rock to Dynamic Loading

In addition to the information addressing the response of soil and rock to dynamic loading presented in FSAR Section 2.5.4.7, the applicant also referred to FSAR Sections 2.5.3 and 2.5.2.5 for discussions of the capable tectonic fault sources and site response analyses and the development of the GRMS, respectively. FSAR Section 2.5.4.4 presents the velocity profiles used in the dynamic site response analysis. Since it was not possible to obtain undisturbed samples from soil layers S-2 and S-3, the applicant assumed dynamic soil properties from the literature cited by the applicant. The staff reviewed the applicant's assumed shear modulus and damping ratio relationships used to perform the site response analysis to obtain the GMRS. The staff reviewed the two sets of EPRI curves, Peninsula Range (PR) and Soft Rock (SR) that the applicant used to represent the range of soft rock behavior in the cemented soil layers S-2 and S-3, and also in the low velocity zone encountered in the Avon Park limestone between an El. of -48 to -67 m (-160 and -220 ft). Soil layer S-1 was not considered as it is either partially or completely removed in the vicinity of the nuclear island. The applicant found that using these two different relationships made little difference in the dynamic site response.

The staff reviewed FSAR Section 2.5.2 where the dynamic relationships for the PR and SR dynamic properties were presented. They are reproduced as shown in SER Figure 2.5.4-14 for convenience. In this figure, it is observed that the two rock types cover a wide range strain related behavior. From this and the wide margin between the site-specific GMRS and the AP1000 DCD CSDRS shown in SER Figure 2.5.4-15, the staff concludes that the choice of dynamic properties for soil layers S-2 and S-3 and the low velocity zone encountered in the Avon Park limestone are relatively unimportant to the determination of the GMRS.

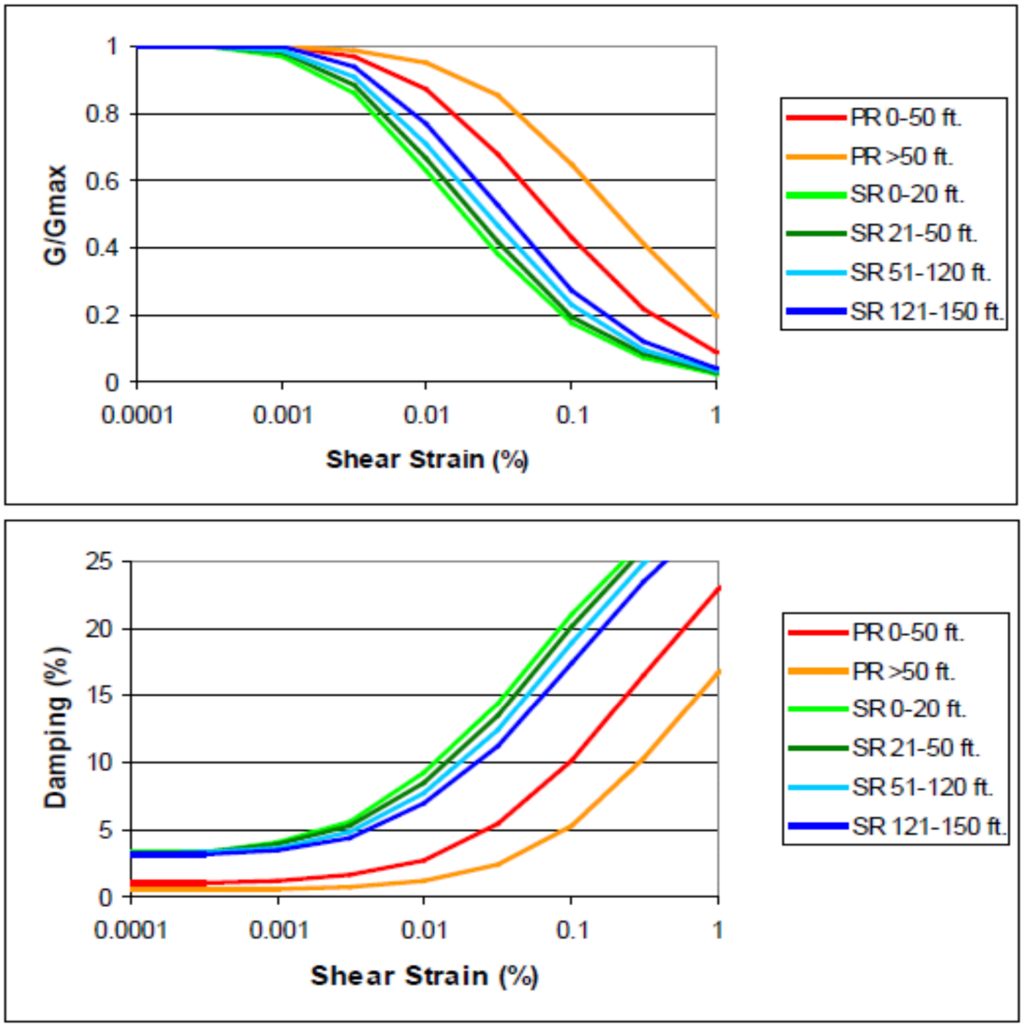


Figure 2.5.4-14. Strain Dependent Shear Modulus and Damping Relationship for Peninsula Rock and Soft Rock (after FSAR Figure 2.5.2-251)

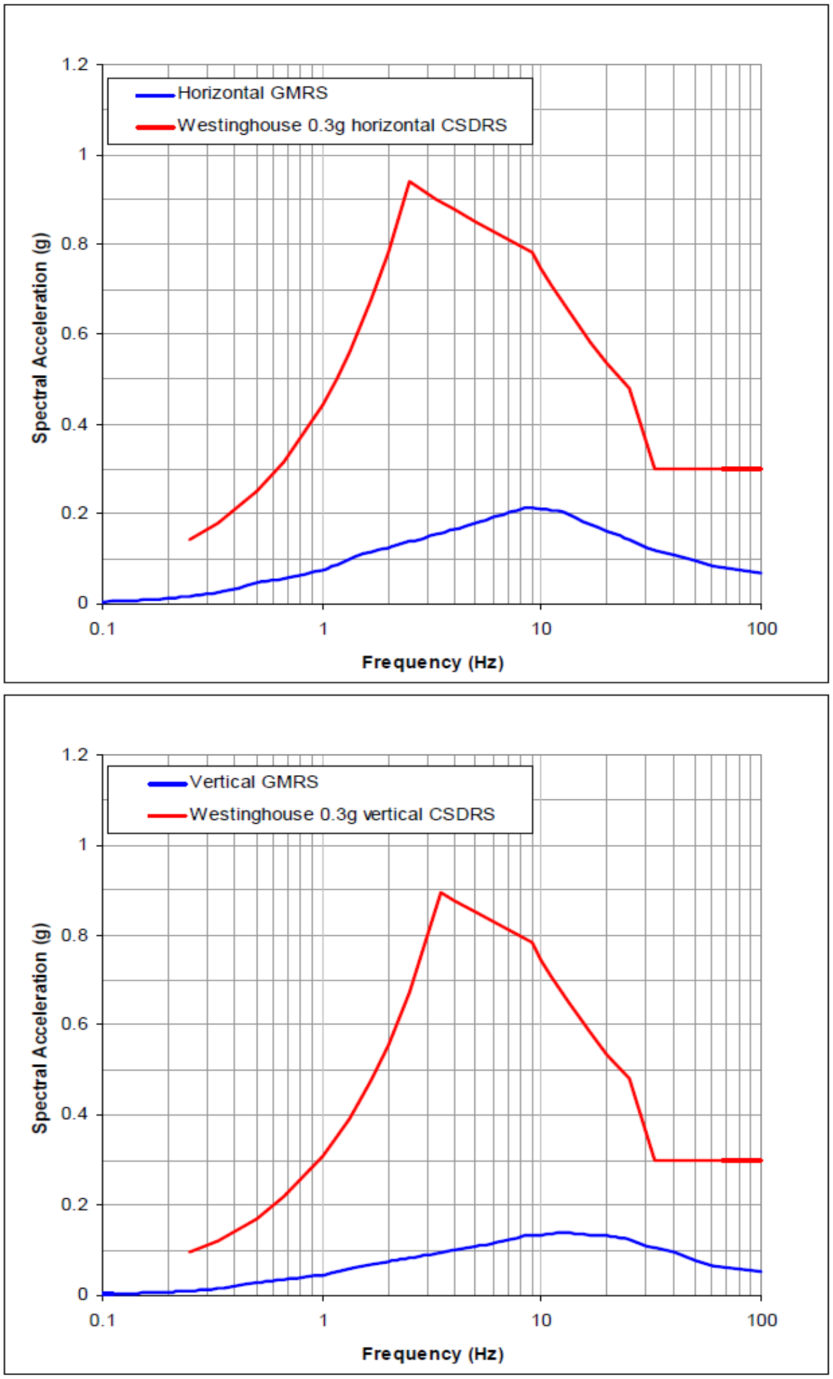


Figure 2.5.4-15. Comparison Between AP1000 Generic Design and Site-specific Response (FSAR Figure 2.5.2-296)

The staff reviewed the requirements for the characterization of the dynamic properties of the soil and rock provided in the AP1000 DCD and concludes that the applicant provided sufficient details to address the requirements of the DCD and satisfy COL Information Item 2.5-6 related to rock dynamic properties. The staff also concludes that the characterization of the dynamic properties of the subsurface materials as described in FSAR Section 2.5.4.7 and related FSAR Sections 2.5.2.5 and 2.5.4.4 forms an adequate basis for the assessment of the response of soil and rock to dynamic loading at the LNP site and meets the requirements of 10 CFR Part 50, Appendix A, GDC2, and Appendix S; and 10 CFR 100.23.

2.5.4.4.8 Liquefaction Potential

In FSAR Section 2.5.4.8, the applicant presented the results of its liquefaction analysis. Because both the Avon Park limestone and the RCC bridging mat are not prone to liquefaction, the applicant stated that liquefaction cannot occur below the nuclear island. However, the applicant found that liquefaction can occur in random zones within the overburden soils, primarily layer S-1, outside the limits of the diaphragm wall. The applicant stated that the random zones of soil with a low factor of safety against liquefaction do not adversely impact nuclear island sliding stability as those zones are isolated and negligible and are generally outside the wedge of soil that resists sliding. More importantly, the applicant concluded analyses by Westinghouse demonstrate that the passive resistance of the backfill is not required for sliding stability. Sliding stability of the nuclear island is evaluated in SER Section 3.8.5.

2.5.4.4.8.1 Liquefaction of Soils Beyond the Diaphragm Wall

The staff reviewed the results of the liquefaction analysis and noted that the applicant identified some foundation materials within the overburden outside the diaphragm wall that are considered to be liquefiable (SF≤1.1) during a SSE event. The staff also noted that these materials appear in isolated areas, some areas designated for removal and replacement with non-liquefiable engineered fill. The staff further notes that these soils are outside the limits of the reinforced concrete diaphragm wall, and would therefore not completely relieve at-rest pressures acting against the nuclear island. Perhaps more importantly, and as discussed earlier with respect to the CLSM, site-specific ground motions are inadequate to cause displacements that would require development of passive pressures, thereby reducing the need for a stable backfill. In order to provide NRC staff reviewing Sections 3.7 and 3.8 with the extent of the liquefiable zones, the staff requested additional information regarding potentially liquefiable soils in RAI 2.5.4-19.

In its June 23, 2009, response to RAI 2.5.4-19, the applicant referred to FSAR Section 2.5.4.8.5, which states that the random zones of soils with a low FS against liquefaction are irrelevant to the sliding stability issue because the zones are isolated and negligible, not required to provide passive resistance that prevents sliding of the nuclear island during the SSE, and/or replaced with non-liquefiable material. The applicant stated that it evaluated the sliding stability of the nuclear islands in a linear static analysis and calculated a FS against sliding of 1.7 irrespective of the passive resistance of the backfill surrounding the nuclear island.

In the September 3, 2009, supplemental response to RAI 2.5.4-19, the applicant addressed overturning as well as sliding and stated that there is no passive pressure required to maintain stability against overturning. The applicant also proposed changes to update FSAR Sections 2.5.4.5.4 and 2.5.4.8.5, LNP COL FSAR Table 2.0-201, and Part 10 of the COL application, Appendix B, Table 3.8-2. In an additional supplement to the response to RAI 2.5.4-19, dated November 5, 2009, the applicant described additional changes made to update FSAR Section 14.3.3.2 that change the minimum coefficient of friction to resist sliding from 0.7 to 0.55.

The staff reviewed the responses to RAI 2.5.4-19 and determined that a review of the linear analysis the applicant performed was needed before the staff could conclude that liquefaction of the backfill was irrelevant to the sliding stability of the nuclear island. Accordingly, in RAI 2.5.4-25 the applicant was requested to provide the linear analysis for the staff's review of LNP COL FSAR Section 3.8.5.

Prior to receiving the response to RAI 2.5.4-25, the applicant responded in a September 3, 2009, supplemental response to RAI 2.5.4-19, that the Westinghouse non-linear sliding analysis discussed in the Westinghouse response to AP1000 RAI TR85-SEB1-10R2 was the sole basis to conclude that isolated pockets of liquefiable zones will not affect the sliding stability of LNP Units 1 and 2. In addition, the applicant voided the site-specific calculation for sliding stability referenced in its original June 23, 2009, response to RAI 2.5.4-19 as it was no longer necessary to support the conclusions.

Subsequent revisions to AP1000 RAI-TR85-SEB1-10R2 resulted in acceptance of AP1000 RAI-TR85-SEB1-10R4, where staff concluded that sliding stability and overturning stability was not dependent on passive resistance of the soil backfill. The staff's evaluation is in NUREG-1793 and its supplements.

In its revised June 8, 2010, response to RAI 3.8.5-3, the applicant prepared plan and profile drawings showing the locations of the liquefied zones to answer questions related to LNP COL FSAR Section 3.8.5 stability concerns. This response was directed to the question of the impact on lateral stability of liquefiable soils surrounding the drilled piers. Evaluation of both the sliding stability and lateral stability of drilled piers are reviewed in SER Sections 3.7 and 3.8.5. The applicant's responses to RAIs 3.8.5-3 and 3.8.5-7 included proposed revisions to LNP COL FSAR Section 2.5.4.5 and Section 2.5.4.8.5 to add information about the liquefied zones. The NRC finds these changes acceptable. Because the applicant has provided the details requested in response to RAI 3.8.5-3, the staff considers RAIs 2.5.4-19 and 2.5.4-25 resolved. The incorporation of changes in a future revision to the LNP COL FSAR is being tracked as **Confirmatory Item 2.5.4-1**.

Resolution of Confirmatory Item 2.5.4-1

Confirmatory Item 2.5.4-1 is an applicant commitment to update section 2.5.4 of its FSAR. The staff verified that LNP COL FSAR Section 2.5.4 was appropriately updated. As a result, Confirmatory Item 2.5.4-1 is now closed.

2.5.4.4.8.2 Revised Liquefaction Analysis for Proposed Backfill

The staff observed that the liquefaction analysis did not include the engineered backfill to be placed between the existing site grade at an El. of 13.1 m (43 ft) to the final plant grade at an El. of 15.5 m (51 ft). In RAI 2.5.4-24, the staff asked the applicant to update the liquefaction evaluations to include the planned backfill.

In its January 19, 2010, response to RAI 2.5.4-24, the applicant presented revised liquefaction evaluations for the modified soil profile, to include the added overburden, and re-calculated the FS. The applicant again followed the guidance of RG 1.198 with respect to calculation of the liquefaction potential and identified the zones for which it calculated the low or intermediate FSs. The calculations included data from the boreholes completed as part of the offset boring program discussed in SER Section 2.5.4.4.3. The applicant's response included replacement tables for FSAR Tables 2.5.4.8-202A and 2.5.4.8-202B that include borehole data from the offset boring program (Tables RAI 2.5.4-24-1 and RAI 2.5.4-24-2, respectively). The applicant concluded that the results of the liquefaction analysis are consistent with the earlier conclusions that liquefaction is confined to isolated pockets.

The staff reviewed the liquefaction analysis results and concludes that in addition to the previously identified zones of liquefaction some additional zones will liquefy that were not identified in the applicant's initial analysis. However, as noted earlier, liquefaction will not impact the foundation of the nuclear island as it is founded on a 10.7 m (35 ft) thick RCC bridging mat resting on the Avon Park limestone, neither of which is liquefiable. Additionally, the staff notes that neither the CLSM backfill immediately surrounding the nuclear island, nor the densely compacted engineered fill that brings the site to plant grade, have the potential for liquefaction. The staff considers RAI 2.5.4-24 resolved. The incorporation of Tables RAI 2.5.4-24-1 and RAI 2.5.4-24-2 in a future FSAR revision is being tracked as **Confirmatory Item 2.5.4-2**.

Resolution of Confirmatory Item 2.5.4-2

Confirmatory Item 2.5.4-2 is an applicant commitment to update Section 2.5.4 of its FSAR. The staff verified that LNP COL FSAR Section 2.5.4 was appropriately updated. As a result, Confirmatory Item 2.5.4-2 is now closed.

The overburden soil layer S-1 that is subject to liquefaction is outside the limits of the 1.06 m (3.5 ft) thick reinforced concrete diaphragm wall, and is partially removed and replaced during construction of the non-safety related structures. The staff notes that limited zones of liquefaction of natural soils occurs in isolated areas surrounding the nuclear island and surrounding some of the drilled pier locations that support the Turbine, Annex and Radwaste

Buildings. To address the liquefaction concerns, the applicant has designed a drainage system consisting of 6 inch diameter vertical drains capped by a 2 ft thick horizontal drainage blanket. The purpose of the drainage system is to relieve the buildup of pore water pressure in the potentially liquefiable zones during earthquake shaking. The pore water pressure relief prevents liquefaction from occurring.

The staff reviewed the design of the drainage system and concludes that the addition of the drainage system, fully penetrating 6 in diameter relief wells, discharging into a 2 ft thick horizontal drainage blanket, has effectively eliminated the liquefaction concerns. The staff further concludes that eliminating the potential for liquefaction preserves the lateral support at the below ground nuclear island walls and at the drilled pier locations.

2.5.4.4.8.3 Liquefaction Potential of CEUS SSC and Seismic Margins Analysis

To evaluate the seismic hazard at LNP site against the new hazard calculation requested by NRC RAI Letter 108, the applicant provided a liquefaction potential assessment using the CEUS SSC model (NUREG-2115) in its FSAR Section 2.5.4.8.7. The staff's detailed review of the applicant's CEUS SSC liquefaction potential evaluation is documented in Subsection 20.1.4.5 of this SER. Based on its review, the staff concludes that the liquefaction evaluations based on the updated EPRI-SOG (design basis) ground motions bound those from the CEUS SSC ground motions.

For the purpose of seismic margins analysis, the applicant also assessed liquefaction potential for ground motions in excess of the site responses corresponding to the GMRS and PBSRS in its FSAR Section 2.5.4.8, and performed sensitivity analysis of the median centered liquefaction potential for 10^{-5} UHRS in its FSAR Section 2.5.4.8.6. The staff's detailed review of the applicant's site-specific seismic margins analysis for liquefaction potential is documented in Subsection 20.1.7.5 of this SER. Based on its review, the staff concludes that the applicant's assumed ground motion based on EPRI-SOG 10^{-5} UHRS for seismic margin considerations is conservative, and concludes that the locations and elevations of hypothesized liquefaction based on 10^{-5} UHRS are almost identical with that based on the design basis.

2.5.4.4.8.4 Conclusion for Liquefaction Potential

Based upon its review of LNP COL FSAR Section 2.5.4.8, the staff concludes that no liquefaction can occur below the nuclear island as the RCC bridging mat and Avon Park formation are both non-liquefiable. The staff further concludes that the site-specific analysis provides an adequate basis to resolve COL Information Item 2.5-9. The staff notes that with the addition of the drainage system in the nonsafety related structure areas, where liquefaction was predicted to occur in the unconsolidated sand layers, liquefaction will be effectively eliminated. The staff therefore concludes that because there is no requirement for passive resistance of the backfill, and because liquefaction is eliminated by the presence of the drainage system, widespread liquefaction of the natural soils surrounding the diaphragm wall and drilled piers will not occur, and potential adverse impacts to the stability of the nonsafety-related structures is effectively controlled. NUREG-1793 and its supplements provide the NRC staff's evaluation of the sliding stability of the Westinghouse AP1000 design indicating no passive resistance requirement for backfill. The review and evaluation of the stability of the drilled piers supporting the seismic Category II and nonsafety-related structures are presented in Sections 3.7.2 and 3.8.5 of this SER.

The staff concludes that the liquefaction analysis described in LNP COL FSAR Section 2.5.4.8 forms an adequate basis for the assessment of the potential for liquefaction at the LNP site and meets the requirements of 10 CFR Part 50, Appendix A, GDC 2, and Appendix S; and 10 CFR 100.23.

2.5.4.4.9 Earthquake Site Characteristics

LNP COL FSAR Section 2.5.4.9, "Earthquake Site Characteristics" refers to FSAR Section 2.5.2 for a detailed discussion of the GMRS. A detailed evaluation of FSAR Section 2.5.4.9 is presented in SER Section 2.5.2.4.

2.5.4.4.10 Static Stability

As part of its review of FSAR Section 2.5.4.10, the staff considered the determination of the bearing capacity, settlement and earth pressures at LNP Units 1 and 2. The following sections discuss these determinations of static stability in greater detail.

2.5.4.4.10.1 Bearing Capacity

The staff reviewed the determination of the bearing capacity at the LNP Units 1 and 2 site, including the information provided to resolve COL Information Item 2.5-10 verifying that the Avon Park limestone is capable of supporting the maximum bearing reaction determined from the analyses described in DCD Appendix 3G of 426 kPa (8,900 psf) static loading and described in LNP COL FSAR Section 3.7.2.4.1.6 of 1,149 kPa (24,000 psf) on soft rock under all combined loads, including the site-specific SSE.

The applicant performed the bearing capacity analyses using both FEM analysis methods and closed form solutions based on plasticity theory to determine the bearing capacity of the Avon Park limestone.

2.5.4.4.10.2 FEM and Closed Form Solutions for Bearing Capacity

Due to the complexity of the rock profile at the LNP site, including possible karst features, the applicant used FEM analyses to confirm bearing capacity results obtained using bearing capacity equations based on plasticity theory. The staff asked the applicant in RAI 2.5.4-2a to provide a detailed explanation of how variability within the supporting rock profile was modeled in the FEM analysis. RAI 2.5.4-2b asked the applicant to describe the FEM results, and RAI 2.5.4-2c asked the applicant to describe how it determined the rock mass properties for use in the USACE bearing capacity equations. (USACE EM 1110-1-1905, 1992)

In its November 20, 2008, response to these three parts of RAI 2.5.4-2, the applicant stated that the layered rock modeled for the FEM analysis consisted of three layers at LNP Unit 1 and four layers at LNP Unit 2 based on the geophysical test results, field data gathered during rock coring, and results of laboratory strength tests. The applicant utilized the SAP2000 software for the FEM analysis to generate a model of the foundation.

The applicant determined the rock mass strength parameters, cohesion and friction angle, from the Hoek-Brown criteria and used these parameters as input in the FEM and bearing capacity equations. The applicant modeled the potential voids in the Avon Park limestone by assuming a range of cavity sizes and assigning zero stiffness to the voids. Void sizes ranged from 3 m (10 ft) wide slots across the entire footprint to 3 and 6 m (10 and 20 ft) cubes located at various critical elevations and positions beneath the base of the RCC and below the bottom of the grouted zone.

The staff reviewed the results of the FEM approach and noted its primary advantage is that voids could be included in the model. The staff noted that the multiple FEM analyses the applicant performed for various cases, with design voids located at various positions below the RCC bridging mat, resulted in calculated FS of at least 3.0.

The applicant also provided additional information on the bearing capacity determinations using the USACE equation. The applicant calculated the bearing capacity for the local and general shear failure cases for LNP Units 1 and 2. For the static analysis, the applicant compared the ultimate bearing capacity to the average bearing pressure to calculate the FSs of 7.6 and 5.7 for the general and local shear failure cases at LNP Unit 2, and 7.2 and 5.3 for the general and local shear failure cases at LNP Unit 1. In the dynamic analysis, the applicant compared the ultimate bearing capacity at the bottom of the RCC bridging mat to the dynamic bearing demand. The applicant determined that the FSs against failure during the SSE were greater than 2.5 for the general shear failure condition.

The staff concludes that the two approaches, FEM and bearing capacity equations, yield factors of safety that are in general agreement with one another and are greater than or equal to factor of safety criteria for nuclear power plants, FS of 3 for the static case, and 2 for the dynamic case. In the finite element analysis, which allowed for the inclusion of postulated voids below the nuclear island, the applicant assumed conservatively sized potential void sizes greater than actual voids sizes based on the field data resulting in conservative assumptions used in the engineering analyses. Accordingly, RAIs 2.5.4-2a through 2.5.4-2c are resolved.

2.5.4.4.10.2.1 Bearing Capacity Sensitivity Analysis Using Closed Form Solutions

The staff reviewed the results of the bearing capacity analysis performed using the bearing capacity equations. In RAI 2.5.4-7b, the staff asked the applicant to describe any sensitivity analyses, which considered variations in the rock mass parameters determined from a statistical analysis of the UCS.

The applicant presented results where it calculated the FS against bearing capacity failure within the Avon Park Formation using three methods: the USACE (1992) method, Hoek, E., et al. (2002) method, and Serrano-Otalla (1994) method. Each of these methods considered three sets of strength parameters based on the mean, median and 84th percentile UCSs of the Avon Park limestone. Based on the results shown in SER Table 2.5.4-5, the staff concluded that the FSs against bearing capacity were adequate. SER Table 2.5.4-5 shows the FS results of the sensitivity analyses were approximately 3.0 for the mean, median and lower bound strength parameters for the general bearing capacity case.

The staff reviewed the bearing capacity sensitivity analyses and performed its own confirmatory analyses. The confirmatory analyses included confirmation that the rock mass properties were representative of the in situ conditions. The staff used the RocLab 1.031 computer program to confirm the rock mass Mohr-Coulomb strength parameters, friction and cohesion, indicated in SER Table 2.5.4-5 for LNP Unit 2, Case 1, mean and lower bound UCS strength values. Using the USACE bearing capacity computer program CBEAR, based on EM 1110-1-1905, the staff determined the FS for LNP Unit 2, Case 1, lower bound UCS values, to be 2.9. This is in agreement with the applicant as shown in SER Table 2.5.4-5. Since the staff reproduced the applicant's results in its confirmatory analyses, the staff concluded that the results presented in SER Table 2.5.4-5 were reliable and the bearing capacity of the foundation rock at the LNP Units 1 and 2 was acceptable.

Table 2.5.4–5. Bearing Capacity Sensitivity Results (Table RAI 2.5.4.7-1)

		North (LNP 2)						South (LNP 1)					
		I*			II**			I			II		
		Mean UCS	Median UCS	Lower bound UCS	Mean UCS	Median UCS	Lower bound UCS	Mean UCS	Median UCS	Lower bound UCS	Mean UCS	Median UCS	Lower bound UCS
Rock Mass Properties	Unit Weight, kg/m^3 (pcf)	2,013 (125.7)			1,890 (118.0)			2,116 (132.1)			2,002 (125.0)		
	Cohesion, kPa (ksf)	201 (4.2)	158 (3.3)	90.9 (1.9)	143 (3.0)	114 (2.4)	71.8 (1.5)	167 (3.5)	143 (3.0)	86.1 (1.8)	153 (3.2)	138 (2.9)	86.1 (1.8)
	Friction Angle, degrees	20.0	18.3	14.8	16.3	14.8	11.6	20.3	19.2	15.8	15.5	14.8	11.9
USACE (1996) General Shear Failure	Ultimate Bearing capacity, kPa (ksf)	3,662 (76.5)	2,896 (60.5)	1,790 (37.4)	4,184 (87.4)	3,490 (72.9)	2,451 (51.2)	3,543 (74.0)	3,016 (63.0)	1,915 (40.0)	4,634 (96.8)	4,280 (89.4)	3,078 (64.3)
	FS	6.0	4.8	2.9	6.2	5.2	3.6	5.8	5.0	3.2	6.2	5.7	4.2
USACE (1996) Local Shear Failure	Ultimate Bearing Capacity, kPa (ksf)	2,743 (57.3)	2,078 (43.4)	1,158 (24.2)	-	-	-	2,599 (54.3)	2,145 (44.8)	1,235 (25.8)	-	-	-
	FS	4.5	3.4	1.9	-	-	-	4.3	3.5	2.0	-	-	-
Hoek et al. (2002)	Ultimate Bearing Capacity, kPa (ksf)	3,614 (75.5)	2,834 (59.2)	1,723 (36.0)	3,940 (82.3)	3,246 (67.8)	2,240 (46.8)	3,868 (80.8)	3,164 (66.1)	1,915 (40.0)	4,337 (90.6)	3,983 (83.2)	2,805 (58.6)
	FS	6.0	4.7	2.8	5.9	4.8	3.3	6.4	5.2	3.1	5.8	5.3	3.8
Serrano-Otalla (1994)	Ultimate Bearing Capacity, kPa (ksf)	5,362 (112.0)	4,036 (84.3)	2,259 (47.2)	4,893 (102.2)	3,974 (83.0)	2,523 (52.7)	5,798 (121.1)	4,591 (95.9)	2,552 (53.3)	5,305 (110.8)	4,802 (100.3)	3,184 (66.5)
	FS	8.8	6.6	3.7	7.3	5.9	3.8	9.5	7.6	4.2	7.1	6.4	4.3

*I refers to the bearing capacity at the top of the Avon Park Formation NAV-1 for (LNP2) and SAV-1 for (LNP1).
**II refers to the bearing capacity at the top of the lower strength zones NAV-3 (LNP2) and SAV-2 (LNP1).

Based on the FEM analyses and the USACE bearing capacity equation solution results, the staff concludes that the Avon Park limestone has an adequate margin of safety for the static and dynamic loads that will be imposed by the RCC bridging mat and nuclear island under both static and dynamic cases, and the bearing capacity meets or exceeds the bearing capacity criteria set forth in the AP1000 DCD. Accordingly, RAI 2.5.4-7b is resolved.

2.5.4.4.10.3 Settlement

The staff focused its review on the calculations of total and differential settlement for the nuclear island and the surrounding seismic Category II and nonsafety-related structures. The staff reviewed: (1) the effect of voids below the grouted zone on settlement; (2) the effects of continuous soft bedding layers on settlement; and (3) the effect of spatial variability in the limestone layer stiffness across the site on settlement. The staff issued the following RAIs prior to the completion of the offset boring program that the applicant performed to characterize the materials in the no recovery zones and that is discussed in detail in SER Section 2.5.4.4.3.

2.5.4.4.10.3.1 Effect of Voids at Depth on Settlement

During the review of the boring logs, the staff noted that very few borings went deeper than an El. of -45.7 m (-150 ft). In RAI 2.5.4-3, the staff asked the applicant to discuss the basis for the conclusion that larger voids do not exist below an El. of -45.7 m (-150 ft). The staff also requested that the applicant provide a sample settlement calculation.

In its November 20, 2008, response to RAI 2.5.4-3, the applicant characterized the karst features in the site vicinity as solution channels in the Avon Park limestone oriented along near-vertical fractures with cavities developing as the fracture walls dissolve. The applicant cited four borings that extended to an El. of -137 m (-450 ft), and an additional 28 borings that extended between an El. of -45.7 and 83.8 m (-150 and -275 ft), and concluded that these borings support the evaluation of karst features described in the FSAR. Finally, the applicant stated that engineering analyses incorporating a conservatively sized void of 6 by 6 m (20 by 20 ft) located below an El. of -45.7 m (-150 ft) demonstrated the safety of the foundation structure.

The applicant based its soil and rock profiles on the geotechnical site investigation data and provided the layered subsurface profiles used in the settlement analyses for LNP Units 1 and 2. The elastic properties of the mass rock were derived from small strain V_s measurements and reduced by 50 percent to account for larger strains. The applicant provided a sample settlement calculation, which concluded that total settlements were less than 0.50 cm (0.2 in).

The staff reviewed the applicant’s response to RAI 2.5.4-3, the borings in Appendix BB, the result of the offset boring program (O-series), and the referenced responses and supplements to other RAIs.

Based on the review, the staff concludes that the karst features appear to occur along vertical fractures and at junctures with horizontal bedding planes in the “plus sign” morphology the applicant described. Additionally, the staff concludes that voids having dimensions greater than the design void of 3 m (10 ft) are not anticipated below an El. of -45.7 m (-150 ft) based on the distribution of voids encountered during the exploration of the site, the increasingly dolomitized nature of the Avon Park limestone with depth, and the reduced ability of downward directed seepage to dissolve limestone as surface water percolates downward. The staff therefore concludes that it is reasonable to assume that larger voids do not exist below an El. of -45.7 m (-150 ft).

The staff also reviewed the FEM analysis results, which show that a 6 m (20 ft) cube void located below the grouted zone, and subjected to the nuclear island static loading, results in the same magnitude settlement, approximately 0.5 cm (0.2 in), as what occurs when no void is present. The staff also noted from the FEM analysis that at two times the static load, the deformation remains essentially linear and the settlement is only 1.3 cm (0.5 in). The staff concludes that this settlement will occur during construction given the stiffness of the Avon Park limestone. The staff finds this predicted settlement acceptable, because it is within the AP1000 DCD limits. Accordingly, RAI 2.5.4-3 is resolved.

2.5.4.4.10.3.2 Settlement Sensitivity to Variations in Elastic Modulus

The staff reviewed the settlement sensitivity to variations in assumed elastic modulus, postulated embedded soft layers, and zones of higher/lower RQDs.

2.5.4.4.10.3.2.1 *Variation in Assumed Elastic Modulus*

The staff reviewed the V_s profiles and observed that some variability from the mean exists, particularly in SAV-1 and NAV-1. In RAI 2.5.4-7c, the staff asked the applicant to describe any settlement sensitivity analyses performed that accounted for variations in the stiffness of the average properties assumed for the layered Avon Park limestone.

The applicant presented the results of a settlement sensitivity analysis varying the stiffness of the Avon Park formation by reducing the mean elastic modulus by one-third, one-half and one standard deviation. The applicant reported that settlements computed by the sensitivity analysis remained well within the range of allowable settlements. Later, in a supplemental response, the applicant compared the properties obtained from the offset boring program described in SER Section 2.5.4.4.3 with those assumed for the sensitivity analyses and confirmed the conservatism of the elastic moduli used in the sensitivity analyses.

The staff reviewed the results of the sensitivity analyses and performed confirmatory calculations of the settlements using elastic theory. Once the staff confirmed that the V_s results accurately represented the in-situ conditions, the staff performed settlement calculations at LNP Units 1 and 2 using profiles provided in response to RAI 2.5.4-3. The relationship for elastic deformation was based on the following equation (Bowles, 1988):

$$\Delta\delta = \sum_{i} \frac{H_i \Delta\sigma_i}{E_{mc}}$$

where:

$\Delta\delta$ is the total elastic settlement
H_i is the thickness of layer i
$\Delta\sigma_i$ is the incremental increase in vertical stress due to foundation loading at the i^{th} layer
E_{mc} is the average constrained elastic modulus derived for large strains from the small strain Vs profiles.

This equation is also used in the American Society of Civil Engineers, “Bearing Capacity of Soils,” which is referenced in NUREG-0800. The settlement analysis was performed to a total depth of 132 m (434 ft). The staff assumed very conservative lower bound elastic modulus values for each of the layers at LNP Units 1 and 2. The elastic modulus values were based on the minimum V_S recorded in the respective layers published in FSAR Table 2.5.4.2-214. As can be seen in this table, the minimum small strain V_S are typically one-half of the average V_S. The elastic modulus values computed from the minimum small strain V_S were then corrected for large strains using the correction factor of 0.5. The staff computed a maximum total settlement of 2.8 cm (1.1 in) for LNP Unit 1 and 2 cm (0.8 in) for LNP Unit 2. Though these calculations were based on conservative E_{mc}’s, the total settlement values are still bounded by the maximum total settlement allowed by the AP1000 DCD, 15.2 cm (6 in). Therefore, the staff concludes that the settlements at LNP Units 1 and 2 are well within the range of acceptable settlements required by the AP1000 DCD. Thus, RAI 2.5.4-7c is resolved.

2.5.4.4.10.3.3 Postulated Embedded Soft Layers

The staff had questions about how the applicant incorporated joints and soft bedding layers into the FEM analyses for settlement. In RAI 2.5.4-8, the staff requested that the applicant describe how it modeled joint patterns and soil filled bedding planes in the FEM analysis for the evaluation of settlement.

In its April 2, 2009, response to RAI 2.5.4-8, the applicant stated it implicitly and explicitly modeled the joints and bedding planes in the FEM analyses and noted that it considered highly conservative shapes, sizes, physical properties and locations of postulated voids. The applicant also stated that its review of the geophysical test results yielded a list of potential soft/infill locations, 13 of which were identified across two or more borings at the LNP site. The applicant modeled these as continuous features to evaluate the total and differential settlement associated with their presence at the LNP site. The applicant did not take any credit for the subsurface improvement that results from grouting between an El. of -7.3 and 30.1 m (-24 and -99 ft). Based on the results of the sensitivity analyses, the applicant concluded that the presence of soft bedding planes would be tolerated by the RCC bridging mat and the settlement would still be within the AP1000 DCD requirements. The applicant further concluded that, based on the highly conservative assumptions used in the sensitivity analysis, an adequate safety margin exists at the LNP site.

On January 19, 2010, the applicant supplemented the response to RAI 2.5.4-8 to include the results of the offset boring program. The applicant compared the conservative properties assumed for the elastic modulus, Poisson’s ratio and unit weight during previous sensitivity analyses with the properties estimated from the results of the offset boring program and concluded that the sensitivity studies were adequately conservative.

The staff reviewed the results of the settlement sensitivity analysis assuming the inclusion of 13 soft continuous 0.3 m (1 ft) thick layers underlying the nuclear island. The staff concludes that the inclusion of these 13 continuous layers within the Avon Park limestone is conservative from the standpoint that they are not present in all borings and are therefore discontinuous. Additionally, the staff concludes that the properties assigned to these layers are conservative based on the results of the boring offset program that demonstrated that the materials are not soil infill, but consist instead of variably weathered limestone. The applicant assigned the soft layers an elastic modulus equivalent to that of loose sand, or about 3 percent of the value assigned to rock layer NAV-1 at LNP Unit 2. The results of the analysis demonstrated that total and differential settlements would only nominally increase, total settlement being less than 1.3 cm (0.5 in). The staff compared the AP1000 DCD settlement criteria of 15.2 cm (6 in) total settlement and/or 1.3 cm (0.5 in) differential settlement in 15.2 m (50 ft) to the total settlement of 1.3 cm (0.5 in) calculated given the conservative assumptions of 13 soft layers, and concludes that the settlement criteria is met. The staff conducted a confirmatory settlement analysis using elastic theory and the applicant's material property assumptions and obtained similar results to those of the applicant. Accordingly, RAI 2.5.4-8 is resolved.

2.5.4.4.10.3.4 Sensitivity and Variability in the Avon Park Formation

For completeness, the staff asked the applicant to determine settlements for the condition where stiffness of the Avon Park limestone varies laterally. In RAI 2.5.4-11, the staff asked the applicant to discuss the settlement sensitivity due to the discontinuous soft bedding planes revealed in the borings.

In its June 23, 2009, response to RAI 2.5.4-11, the applicant used a 3D FEM to perform the sensitivity analysis, which evaluated the settlements considering the static loads and the weight of the RCC bridging mat.

To address the settlement sensitivity to lateral variation in layer stiffness as observed in zones of higher and lower RQDs, the applicant submitted the results of a sensitivity analysis in which it varied the elastic properties across the foundation footprint based on RQD values. The applicant devised two zones for the settlement sensitivity analysis, as shown in SER Figure 2.5.4-16. One zone consists of limestone exhibiting medium to high RQDs of greater than 50 percent, and a second zone consisting of medium to low RQDs of less than 50 percent. The applicant noted that the zoning based on these RQD values created localized zones of softer material surrounded by zones of stiffer material, consistent with the conclusion that soft bedding layers are limited in extent and do not extend across the entire footprint of the LNP site.

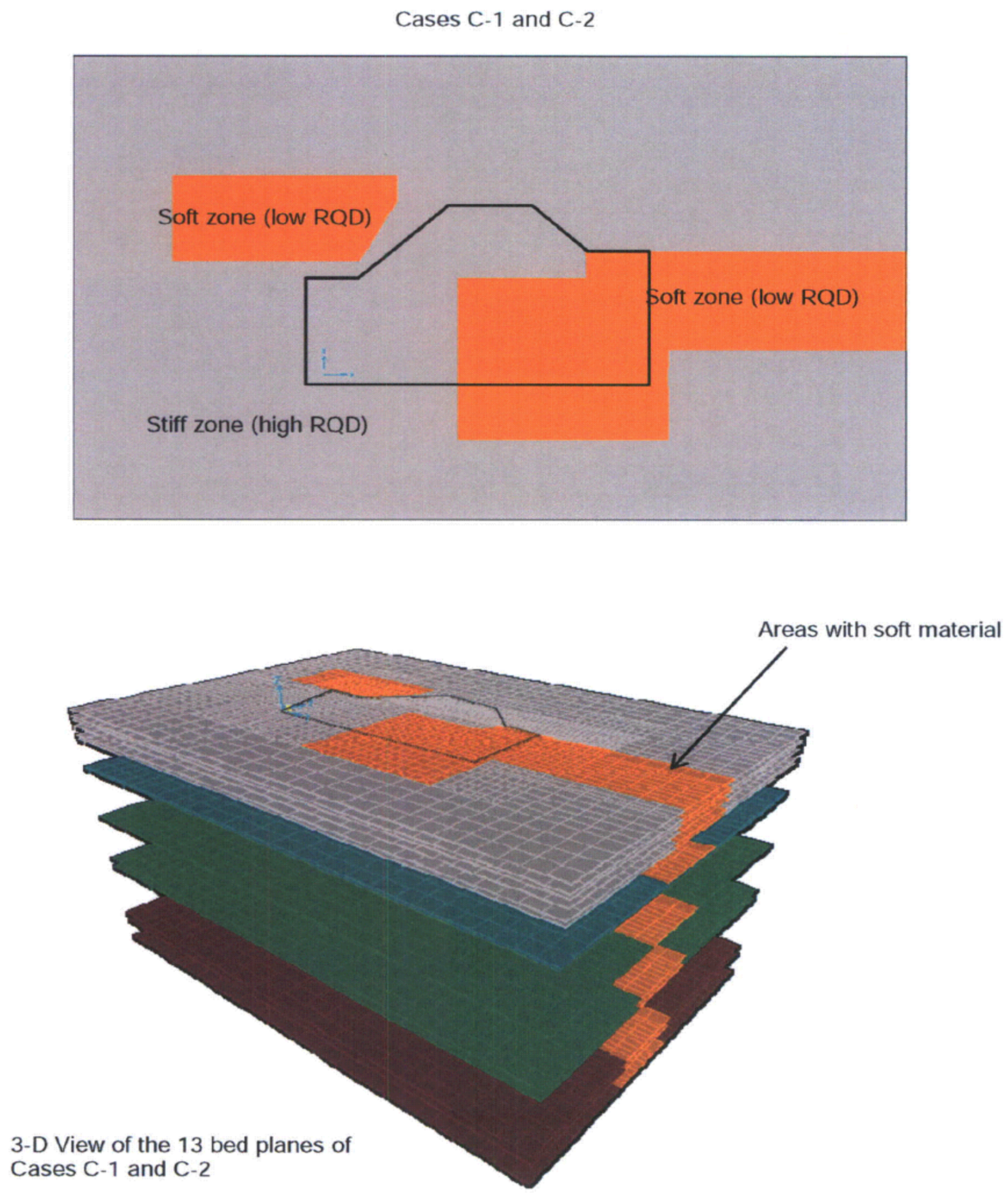


Figure RAI 02.05.04-11-02
Horizontal Variation of Soft Zones (Cases C-1 and C-2)

Figure 2.5.4-16. Distribution of Postulated Soft/Stiff Regions and Thirteen Soft 0.3-m (1-ft) Thick Bedding Layers (RAI 2.5.4-11 Response Figure 2.5.4-11-02)

The applicant considered three cases and compared the results to a base case of no soft zones. The applicant calculated the total and differential settlements for all cases to be less than approximately 1.3 cm (0.5 in). The applicant also noted that the largest total and differential settlements occur when the soft bedding planes were modeled as continuous soft layers.

The staff reviewed the results of the sensitivity analyses the applicant completed. Because the sensitivity analyses considered the lower bound values of elastic modulus for the layered Avon Park limestone profile, horizontal variations in elastic properties suggested by variations in RQD across the site, and postulated soft layers that may exist in the rock profile based on limited data in “no recovery” zones, the staff concludes that the sensitivity analyses are sufficient. Accordingly, the staff concludes that the total and differential settlements are acceptable because they are within the AP1000 DCD limits. Thus, RAI 2.5.4-11 is resolved.

2.5.4.4.10.3.5 Settlement Monitoring

The staff reviewed the applicant’s plans to monitor settlement at the LNP site. In RAI 2.5.4-10, the staff asked the applicant to estimate the settlement beneath the seismic Category II and nonsafety-related structures to observe the magnitude of differential settlement between structures, and describe the monitoring program proposed to ensure that the actual and differential settlements do not exceed the DCD settlement criteria.

In its June 8, 2009, response to RAI 2.5.4-10, the applicant provided a table of the estimated total settlements for the Turbine, Annex, Radwaste, and Diesel Generator Buildings for LNP Units 1 and 2 and noted that these total settlements result in differential settlements within acceptable limits. The applicant also revised the FSAR to describe the installation of settlement benchmarks at the nonsafety-related structures to measure the differential settlement during and after construction.

The staff notes that the AP1000 DCD limits the acceptable total settlement of structures to 15.2 cm (6.0 in), and differential settlement between structures to 7.6 cm (3.0 in). Likewise, differential settlement across the nuclear island foundation mat is limited to 1.3 cm (0.5 in) in 15.2 m (50 ft). The staff notes that the settlement estimates of the structures surrounding the nuclear island range from 0.3 to 0.5 cm (0.1 to 0.2 in). Because the average total settlement for the LNP Units 1 and 2 nuclear islands are 0.5 cm (0.2 in), the staff concludes that the total and differential settlement predictions are well within the allowable limits for total settlement, differential settlement between buildings, and tilt or distortional settlement within the nuclear island basemat. The staff also reviewed the changes to the FSAR, including the description of the installation of settlement benchmarks to measure the differential settlement and concludes that the method of measuring the differential settlement at the LNP site is adequate. Accordingly, RAI 2.5.4-10 is resolved.

2.5.4.4.10.3.6 Modeling Discontinuities in the FEM Analysis

The staff reviewed FSAR Section 2.5.4.1, which describes the fracture patterns at the site. In RAI 2.5.4-23, the staff asked the applicant to explain how it incorporated the information related to the observed local fracture patterns into the 3D FEM analysis. The staff also asked the applicant to: (1) clarify whether more closely-spaced fractures occur in the two outcrops discussed; (2) explain whether the fractures are characteristic of the fracture sets at the site location; and (3) explain how the design analyses account for settlement due to discontinuities.

In its June 23, 2009, response to RAI 2.5.4-23, the applicant used data from the Grout Test Program to confirm that the fracture orientation observed in the field is consistent with the regional orientation. The applicant stated that the fractures are typically less than 3 cm (0.1 ft) in width. The applicant stated that the four different cases modeled in the FEM sensitivity analysis combined the multiple 3 cm (0.1 ft) wide fractures into a 3 m (10 ft) wide fracture or orthogonal fracture set.

As modeled, the applicant stated that the 3.04 m (10 ft) fracture placed through the center of the nuclear island as shown in SER Figure 2.5.4-17 represents 100 fractures of 0.3 cm (0.1 ft) thickness and produces a maximum elastic settlement of 0.68 cm (0.27 in) and a differential settlement of 0.43 cm (0.17 in), which are less than the allowable settlement allowance of the AP1000 DCD.

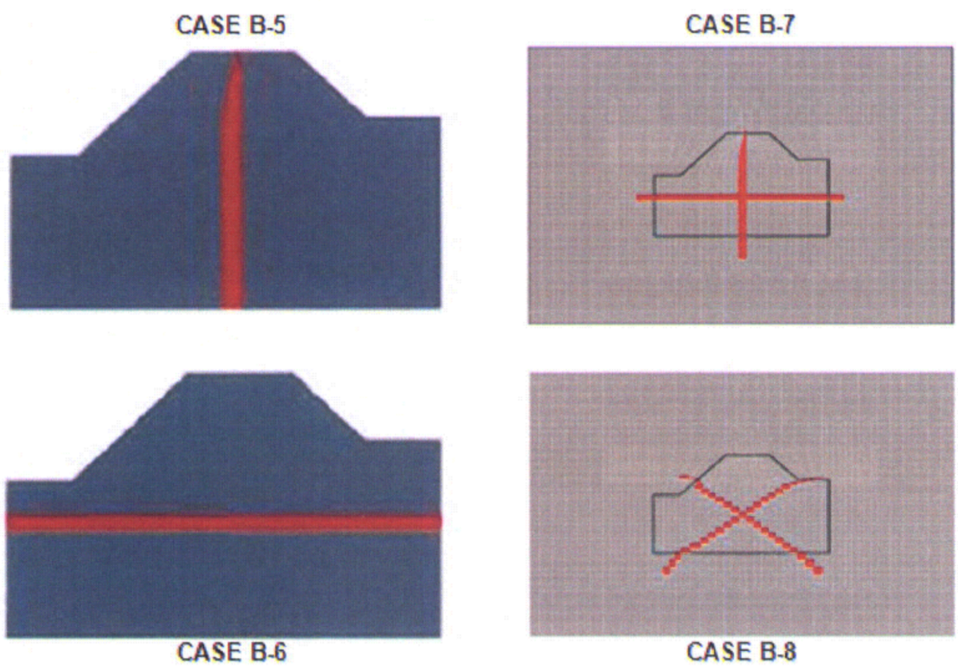


FIGURE RAI 02.05.04-23-1
LOCAL FRACTURE SYSTEMS MODELED IN FEM

Figure 2.5.4-17. Representation of Local Fracture System in Finite Element Method Sensitivity Analyses (RAI Figure 2.5.4-23-1)

The staff reviewed the RAI response discussing the FEM results, and concludes that the aggregation thin 0.3 cm (0.1 ft) orthogonal fractures spaced a minimum of every 5.7 m (19 ft) into a large, orthogonal 3 m (10 ft) wide fracture placed through the center of the nuclear island represents the critical case and is conservative. The staff also notes that the subgrade surface preparation with dental concrete will eliminate all the vertical fractures to a minimum depth of 1.5 m (5 ft). The grouting program will also fill the larger joint openings and bedding plane voids down to El. -30.1 m (-99 ft), leaving little opportunity for large voids to exist within the grouted zone. Because voids greater than 0.9 m (3 ft) in lateral extent were not encountered, the staff concludes that the placement of continuous 3 m (10 ft) wide fracture in the patterns shown in SER Figure 2.5.4-16 are conservative because they are larger and more severe than any single discontinuous void. Accordingly, RAI 2.5.4-23 is resolved.

2.5.4.4.10.4 Lateral Earth Pressures

The staff reviewed FSAR Section 2.5.4.10.4 and FSAR Table 2.5.4.10-205, and determined that a sample calculation was needed in order to complete its review. In RAI 2.5.4-21, the staff asked the applicant to provide sample calculations for both the seismic at-rest and hydrodynamic pressures.

In its June 9, 2009, response to RAI 2.5.4-21, the applicant presented Wood's method (ASCE 4-98, 2000) to calculate the seismic at-rest pressure. The applicant stated that it used Wood's method because the walls are unyielding, which generates greater forces on the wall than those obtained by the Mononobe-Okabe method, which assumes the wall is free to move. A flexible-wall assumption underestimates the dynamic lateral forces generated on a rigid, unyielding wall. Using a Poisson's ratio of 0.3 for the undifferentiated sediments and a thrust factor of 0.98, the applicant concluded that the seismic induced lateral load is 470 kPa (9.83 ksf) and provided a figure showing the dynamic soil pressure resultant force as well as a seismic earth pressure diagram. For the hydrostatic water thrust, the applicant used Westergaard's equation (Westergaard, 1933) and provided a sample calculation and a figure illustrating the hydrostatic pressure.

The staff reviewed the sample calculations and figures and concludes the applicant used conservative material properties and conservative methods in the determination of the static and dynamic lateral earth pressures. Thus, RAI 2.5.4-21 is resolved.

2.5.4.4.10.4.1 Subsurface Instrumentation

In FSAR Section 2.5.4.10.3.5, the applicant addressed the construction and long-term instrumentation monitoring program. The staff considered the details of the construction monitoring plans, including the installation of piezometers to monitor drawdown of the water table and measure piezometric pressures on the bottom of the excavation during excavation and backfilling, heave points to measure heave of the foundation subgrade, and markers on the RCC bridging mat and nuclear island and surrounding structures to measure settlement during construction and until 90 percent of the expected settlement has occurred, or the rate of settlement stops. The staff concludes that long-term settlement will be negligible because of the strength of the foundation materials and the low levels of stress below the RCC bridging mat.

Nevertheless, post construction settlement will be monitored. The long-term monitoring program will be implemented after the construction monitoring program is completed and will monitor any long-term settlement occurring during the life of the structure. The applicant provided a conceptual plan in the FSAR and intended to finalize the instrumentation and monitoring plan during detailed design.

The staff reviewed the applicant's plans to monitor water levels during dewatering and excavation, bottom heave, and settlement of all the structures, and concludes that the applicant's conceptual plan adequately considered the construction features that require monitoring during construction.

2.5.4.4.10.4.2 Resolution of COL Information Items

The staff reviewed FSAR Section 2.5.4.10 and referenced the AP1000 DCD engineering criteria for settlement and bearing capacity. The staff also reviewed responses to related RAIs, and the references cited.

The staff concludes that the bearing capacity of the Avon Park limestone is sufficient to meet both the static and dynamic loading demands of the nuclear island and there is an adequate basis to resolve COL Information Item 2.5-10. The staff concludes that settlement, differential settlement of the nuclear island, and differential settlement between the nuclear island and surrounding structures due to either static or dynamic loading have been thoroughly examined and the estimated settlements are within the criteria set forth in the AP1000 DCD, and that there is an adequate basis to resolve COL Information Item 2.5-12 and COL Information Item 2.5-16. The staff further concludes that the use of Wood's Method to determine the lateral stresses was conservative and there is an adequate basis to resolve COL Information Item 2.5-11. Finally, the staff concludes that the instrumentation planned for monitoring during the construction phase and post-construction for the life of the plant is adequate and appropriate for the features being constructed and that there is an adequate basis to resolve COL Information Item 2.5-13.

2.5.4.4.10.4.3 Conclusion for Static Stability

In FSAR Section 2.5.4.10, the applicant considered the bearing capacity, settlement, lateral stresses, and performance monitoring at the LNP site. Based on the extensive analytical results, the staff concludes that the bearing capacity of the Avon Park limestone is sufficient to meet both the static and dynamic loading demands of the nuclear island. The response to the maximum static loads imposed by the nuclear island and overlying RCC bridging mat on the Avon Park limestone was satisfactory, limiting settlements to approximately 0.51 cm (0.2 in). It was also determined that a FS of 3 exists against bearing capacity failure, with or without a large (6 m (20 ft) cube-shaped) void, located below the grouted zone under the reactor building. Sensitivity analyses using closed form bearing capacity equations indicated acceptable FSs for lower bound material strength assumptions. Elastic settlement sensitivity analyses determined that under the most conservative of assumption of thirteen 0.3-m (1-ft) continuous soft layers located under the footprint of the nuclear island, settlement will be less than 1.3 cm (0.5 in).

NRC staff reviewed FSAR Section 2.5.4.10 and concludes that the applicant developed an accurate assessment of the static stability at the LNP site that addresses COL Information Items 2.5-10 through 2.5-13 and 2.5-16, including the minimum static bearing capacity; earth pressures; static stability of facilities; and subsurface instrumentation. The staff concludes that the information provided with respect to the required bearing capacity of foundation materials is adequate to address Interface Item 2.13. Accordingly, the staff concludes that the applicant's information in FSAR Section 2.5.4.10 forms an adequate basis for the static stability at the site and meets the requirements of 10 CFR Part 50, Appendix A, GDC 2, and Appendix S; and 10 CFR 100.23.

2.5.4.4.11 Design Criteria

Based upon its review of LNP COL FSAR Section 2.5.4.11, including the AP1000 DCD design criteria, methods of analysis the applicant used, and the FS criteria, the staff concludes that the applicant applied good engineering judgment, state-of-the art analytical methods, appropriate design criteria and provided an adequate FS to ensure the safety of SSCs at the LNP site area. The staff concludes that the design values as described in LNP COL FSAR Section 2.5.4.11 form an adequate basis for the design criteria and meet the design values of the AP1000 DCD and the requirements of 10 CFR Part 50, Appendix A, GDC 2, and Appendix S.

2.5.4.4.12 Techniques to Improve Subsurface Conditions

In FSAR Section 2.5.4.12, the applicant summarized the measures that it will implement to improve the subsurface conditions. The applicant planned to grout the Avon Park limestone using grout holes, including inclined grout holes if deemed necessary, in multiple stages and install a reinforced concrete diaphragm wall surrounding the nuclear island to form an impermeable "bathtub" to minimize seepage into the excavation. The applicant will excavate in approximate 3 m (10 ft) depth increments to an El. of -7.3 m (-24 ft), at which point the subgrade will be cleaned, voids backfilled with dental concrete, and surface leveled prior to the construction of the 10.7 m (35 ft) thick RCC bridging mat.

The NRC staff reviewed FSAR Section 2.5.4.12. The applicant plans to remove weak, compressible, severely weathered Avon Park limestone and construct an RCC bridging mat. The staff concludes that the remedial measures the applicant proposed will improve the foundation conditions and provide a uniformly strong base of rock upon which the RCC bridging mat is founded to support the nuclear island. Though meant only to reduce seepage into the excavation during construction, the presence of the diaphragm wall and grouted limestone between an El. of -7.3 and -30.1 m (-24 and -99 ft) will also add to the future stability of the site by reducing the opportunity for future karst development.

Based upon its review of LNP COL FSAR Section 2.5.4.12, the staff concludes that the applicant adequately described its plans for improving and monitoring the subsurface conditions at the LNP site. The staff concludes that the methods of improvement and monitoring plans as described in FSAR Section 2.5.4.12 form an adequate basis for the improvement of subsurface conditions at the site and meet the requirements of 10 CFR Part 50, Appendix A, GDC 2, and Appendix S.

2.5.4.5 *Post Combined License Activities*

There are no post-COL activities related to this section.

2.5.4.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The staff's review confirmed that the applicant addressed the required information relating to the stability of subsurface materials and foundations, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the staff's technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

Based on its review of LNP COL FSAR Section 2.5.4 and the applicant's responses to the RAIs, the staff concludes that the applicant adequately determined the engineering properties of the soil and rock underlying the LNP COL site through its field and laboratory investigations. The staff concludes that the applicant used the latest field and laboratory methods, in accordance with RG 1.132, Revision 2; RG 1.138, Revision 2; and RG 1.198, to determine the required site-specific engineering properties for the LNP site and to ensure that these properties met the design criteria outlined in the AP1000 DCD.

Based on the information in the FSAR, the staff concludes that the subsurface profile underlying the COL site has been properly characterized, that state-of-the-art analytical methods were used with conservative input values to determine factors of safety, and that the applicant considered all aspects of the foundation design that could impact the SSCs. Specifically, the staff concludes that the applicant adequately determined: (1) the soil and rock dynamic properties through its field investigations and laboratory tests; (2) the response of the soil and rock to dynamic loading; (3) the liquefaction potential of the soils; and (4) the static stability, including the bearing capacity, settlement, and lateral earth pressures.

The staff concludes that the applicant provided sufficient information in LNP COL 2.5-5 through LNP COL 2.5-13, and LNP COL 2.5-16 to adequately address the COL information items pertaining to FSAR Section 2.5.4.

The staff concludes that FSAR Section 2.5.4 is acceptable and meets the requirements of 10 CFR Part 50, Appendix A (GDC 2) and Appendix S; and 10 CFR 100.23.

2.5.5 Stability of Slopes

2.5.5.1 *Introduction*

LNP COL FSAR Section 2.5.5 addresses the stability of all earth and rock slopes, both natural and man-made (cuts, fill, embankments, dams, etc.), whose failure, under any of the conditions to which they could be exposed during the life of the plant, could adversely affect the safety of the plant. The following subjects are evaluated using the applicant's data in the FSAR and information available from other sources: (1) slope characteristics; (2) design criteria and design analyses; (3) results of the investigations including borings, shafts, pits, trenches, and laboratory tests; (4) properties of borrow material, compaction and excavation specifications; and (5) any additional information requirements prescribed within the "Contents of Application" sections of the applicable subparts to 10 CFR Part 52.

2.5.5.2 *Summary of Application*

Section 2.5 of the LNP COL FSAR, Revision 9, incorporates by reference Sections 2.5.5 and 2.5.6 of the AP1000 DCD, Revision 19.

In addition, in LNP COL FSAR Sections 2.5.5, the applicant provided the following:

AP1000 COL Information Items

- LNP COL 2.5-14

The applicant provided additional information in LNP COL 2.5-14 to address COL Information Item 2.5-14 (COL Action Item 2.5.5-1), which addresses the static and dynamic stability of site-specific soil and rock slopes with regard to how their failure could adversely affect the nuclear island.

- LNP COL 2.5-15

The applicant provided additional information in LNP COL 2.5-15 to address COL Information Item 2.5-15 (COL Action Item 2.5.6-1), which addresses the static and dynamic stability of site-specific embankments and dams with regard to how their failure could adversely affect the nuclear island.

The applicant developed FSAR Section 2.5.5 for evaluation of slope stability at the LNP site based on information derived from site investigations, geotechnical characterization studies, and excavation and backfill profiles presented in FSAR Sections 2.5.4.1 thorough 2.5.4.5. These investigations and studies included consideration of geologic features and characteristics; site exploration involving soil and rock boring and sampling, groundwater monitoring, in situ testing, laboratory testing, and geophysical surveys.

2.5.5.2.1 Slope Characteristics

FSAR Section 2.5.5 describes the lack of existing permanent slopes, or dams, both natural and man-made, at the LNP site. The applicant stated that the only sloping ground at the LNP site consists of minor elevation changes to accomplish positive drainage away from the nuclear islands. The applicant also stated that the AP1000 does not utilize safety-related dams and that no dams exist that could affect the nuclear islands. The applicant concluded that no permanent slopes or dams exist for which failure would adversely affect the safety-related structures of LNP Units 1 and 2.

2.5.5.3 *Regulatory Basis*

The regulatory basis of the information incorporated by reference is addressed in NUREG-1793 and its supplements.

In addition, the acceptance criteria associated with the relevant requirements of the Commission regulations for the stability of slopes are given in Section 2.5.5 of NUREG-0800.

The applicable regulatory requirements for reviewing the applicant's discussion of stability of slopes are:

- 10 CFR Part 50, Appendix A, GDC 2, as it relates to consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 50, Appendix S, as it applies to the design of nuclear power plant SSCs important to safety to withstand the effects of earthquakes.

- 10 CFR 100.23, provides the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and design of nuclear power plants.

The related acceptance criteria from Section 2.5.5 of NUREG-0800 are as follows:

- Slope Characteristics: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of slope characteristics is acceptable if the section includes: (1) cross sections and profiles of the slope in sufficient quantity and detail to represent the slope and foundation conditions; (2) a summary and description of static and dynamic properties of the soil and rock comprised by seismic Category I embankment dams and their foundations, natural and cut slopes, and all soil or rock slopes whose stability would directly or indirectly affect safety-related and Category I facilities; and (3) a summary and description of groundwater, seepage, and high and low groundwater conditions.

- Design Criteria and Analyses: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of design criteria and analyses is acceptable if the criteria for the stability and design of all seismic Category I slopes are described and valid static and dynamic analyses have been presented to demonstrate that there is an adequate margin of safety.

- Boring Logs: In meeting the requirements of 10 CFR Parts 50 and 100, the applicant should describe the borings and soil testing carried out for slope stability studies and dam and dike analyses.

- Compacted Fill: In meeting the requirements of 10 CFR Part 50, the applicant should describe the excavation, backfill, and borrow material planned for any dams, dikes, and embankment slopes.

In addition, the geologic characteristics should be consistent with appropriate sections from: RG 1.28, Revision 4; RG 1.132, Revision 2; RG 1.138, Revision 2; RG 1.198; and RG 1.206.

2.5.5.4 *Technical Evaluation*

The NRC staff reviewed Section 2.5.4 of the LNP COL FSAR and checked the referenced DCD to ensure that the combination of information presented in the FSAR and the DCD completely represents the required information related to the stability of slopes. The staff's review confirmed that information contained in the application or incorporated by reference addresses the information required for this review topic. NUREG-1793 and its supplements document the results of the staff's evaluation of the information incorporated by reference into the LNP COL application.

The staff reviewed the information in the LNP COL FSAR:

AP1000 COL Information Items

- LNP COL 2.5-14

The NRC staff reviewed LNP COL 2.5-14 in Section 2.5.5 of the LNP COL FSAR, related to the stability of all earth and rock slopes both natural and manmade (cuts, fill, embankments, dams, etc.) whose failure, under any of the conditions to which it could be exposed during the life of the plant, could adversely affect the safety of the plant. The COL information item in AP1000 DCD Section 2.5.5 states:

> Combined License applicants referencing the AP1000 design will address site-specific information about the static and dynamic stability of soil and rock slopes, the failure of which could adversely affect the nuclear island.

With respect to COL Information Item 2.5-14, the applicant stated that there are no soil or rock slopes the failure of which could adversely affect the safety-related structures at the LNP site. The applicant stated that the only slopes consist of minor grading for drainage away from the nuclear islands at LNP Units 1 and 2. The staff reviewed the site plans and concludes that the applicant has appropriately characterized the site conditions. The only sloping boundaries are related to drainage around the nuclear islands and these slopes do not constitute a slope stability concern. The staff concludes that there are no slopes or dams at the site that could adversely affect LNP Units 1 and 2. The staff concludes that the applicant met the criteria of COL Information Item 2.5-14.

- LNP COL 2.5-15

The NRC staff reviewed LNP COL 2.5-15 in Section 2.5.5 of the LNP COL FSAR, related to the stability of embankments and dams, the failure of which could adversely affect the plant. The COL information item in AP1000 DCD Section 2.5.6 states:

> Combined License applicants referencing the AP1000 design will address site-specific information about the static and dynamic stability of embankments and dams, the failure of which could adversely affect the nuclear island.

Regarding COL Information Item 2.5-15, the applicant stated that there are no dams or embankments the failure of which could adversely affect the safety-related structures at the LNP site. The staff considered the results of site investigations, as well as the applicant’s assertion that there are no man-made earthen or rock dams present at the site. The staff concludes that there are no dams or embankments, which might adversely affect Units 1 and 2, and therefore the applicant addressed the criteria of COL Information Item 2.5-15 for the LNP site.

2.5.5.5 *Post Combined License Activities*

There are no post-COL activities associated with this FSAR section.

2.5.5.6 *Conclusion*

The NRC staff reviewed the application and checked the referenced DCD. The NRC staff’s review confirmed that the applicant addressed the required information relating to stability of slopes, and there is no outstanding information expected to be addressed in the LNP COL FSAR related to this section. The results of the NRC staff’s technical evaluation of the information incorporated by reference in the LNP COL application are documented in NUREG-1793 and its supplements.

As set forth above, the applicant presented and substantiated information to establish the stability of all earth and rock slopes, both natural and manmade at the plant site. The staff reviewed the site investigations performed for LNP Units 1 and 2, and the site plans to confirm that there were no slopes or dams that could adversely affect the safe operations of the LNP Units 1 and 2. The staff concludes that the applicant provided sufficient information to addresses COL Information Items 2.5-14 and 2.5-15. The staff concludes that the relevant information presented in LNP COL FSAR Section 2.5.5 is acceptable and meets the requirements of 10 CFR Part 50, Appendix A, GDC 2; 10 CFR Part 50, Appendix S; and 10 CFR 100.23.