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 (PEF) Levy Nuclear Plant (LNP) Units 1 and 2 Final Safety Analysis Report (FSAR) Revision 2, AP1000 Design Control Document (DCD) Revision 17, 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.

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.

2.4.1.1 *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:

COL Information Item

- 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 17 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.

2.4.1.2 *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).

2.4.1.3 *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.[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 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.

2.4.1.3.1 Site and Facilities

Information Submitted by the Applicant

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 13 km (8 mi) inland from the Gulf of Mexico, 4.8 km (3 mi) north of Lake Rousseau, and 15.4 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.

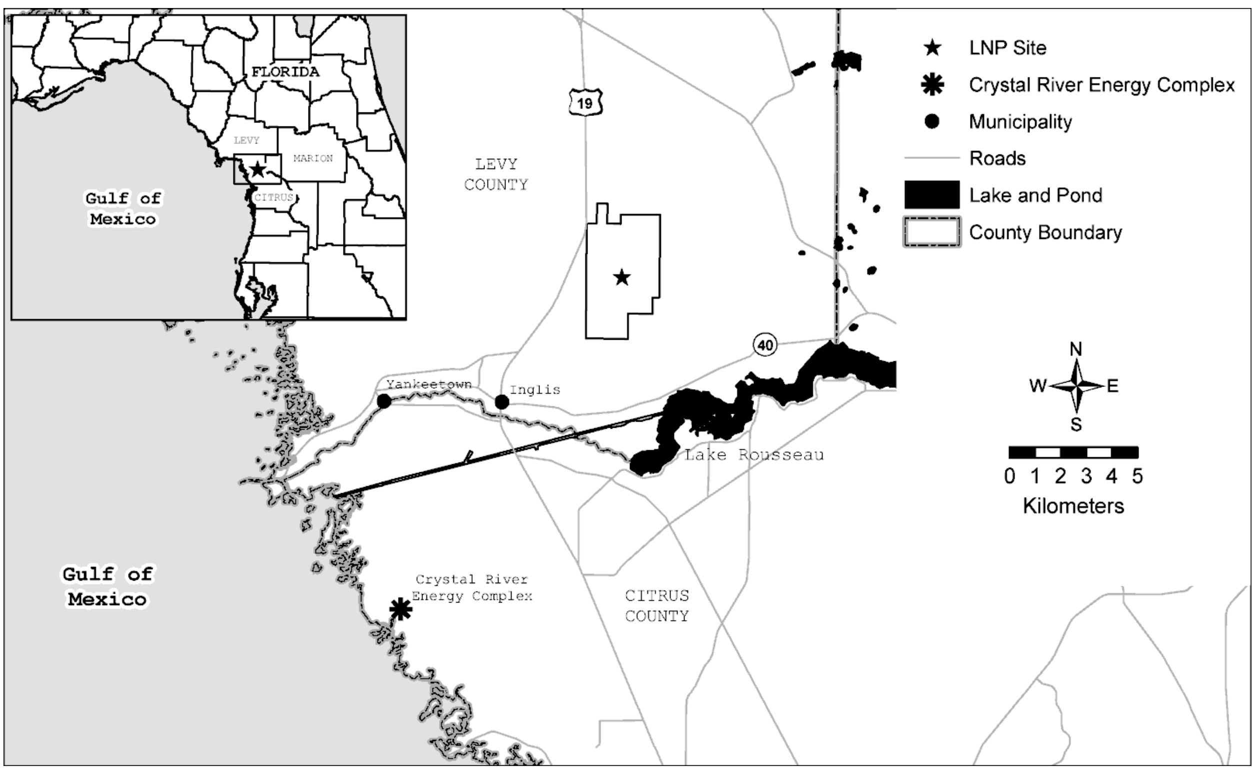


Figure 2.4.1-1. The LNP Site and Surrounding Area

Elevations at the LNP site range from 9.1 to 18.3 m (30 to 60 ft) NGVD29 (National Geodetic Vertical Datum of 1929). The applicant stated that the nominal plant grade would be 15.5 m

[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.

(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 is equivalent to NGVD29. 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 2.

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.

2.4.1.3.2 Hydrosphere

Information Submitted by the Applicant

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.

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.

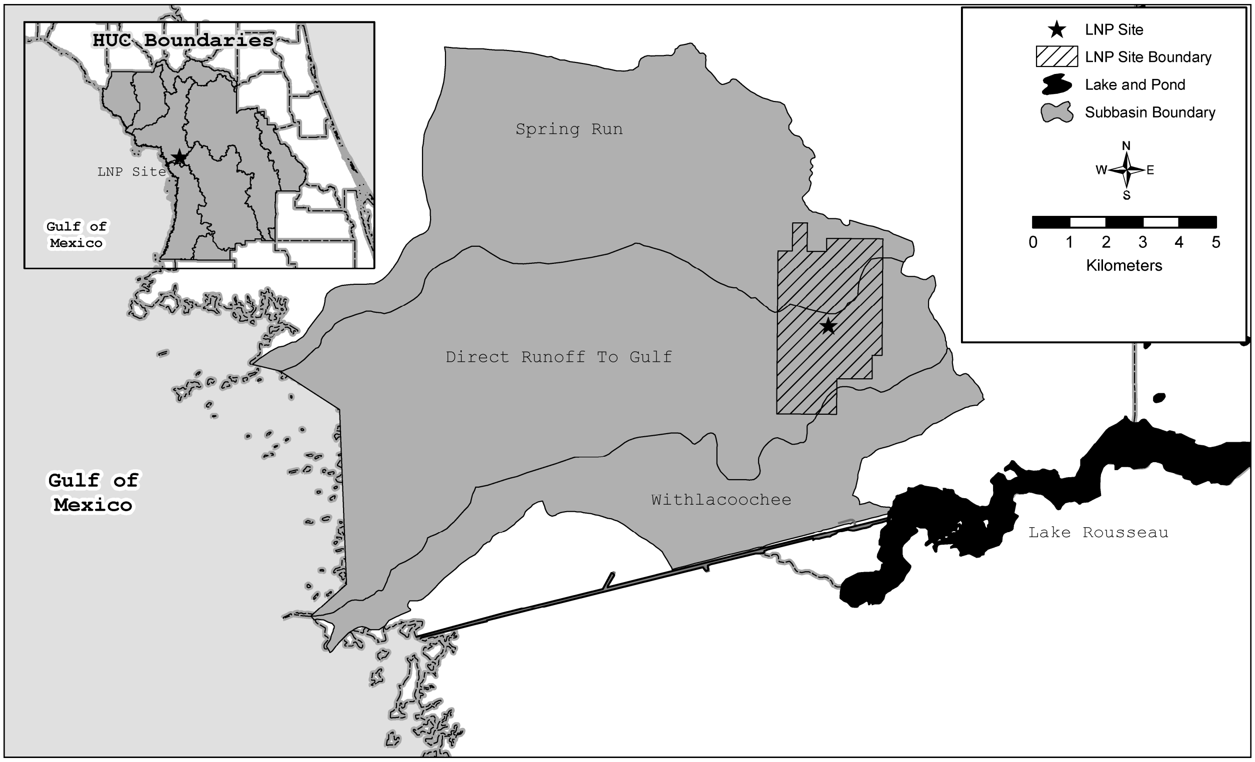


Figure 2.4.1-2. The Subbasins Within Which the LNP Site is Located

Figure 2.4.1-4 shows six U.S. Geological Survey (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) MSL, 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 183 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.

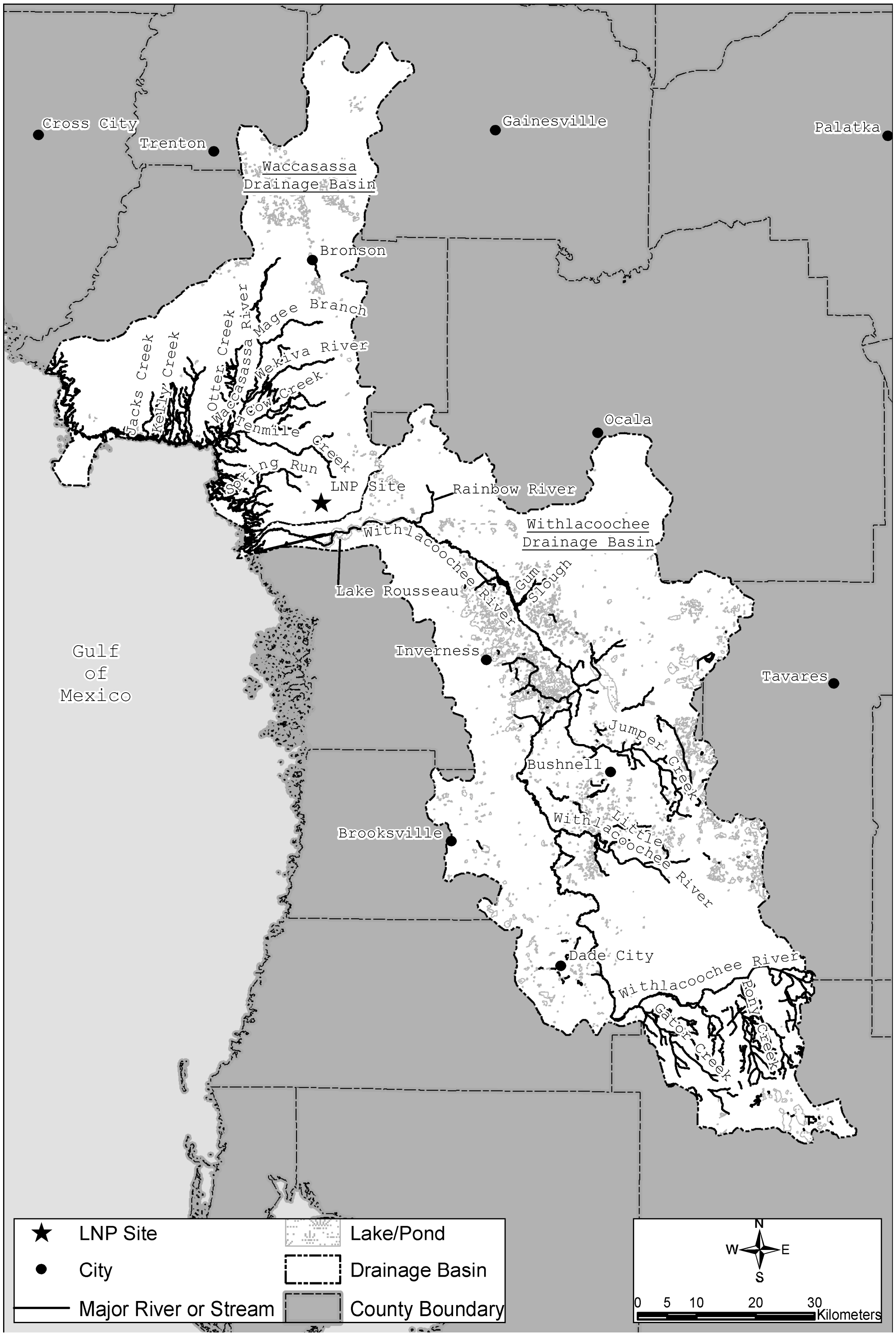
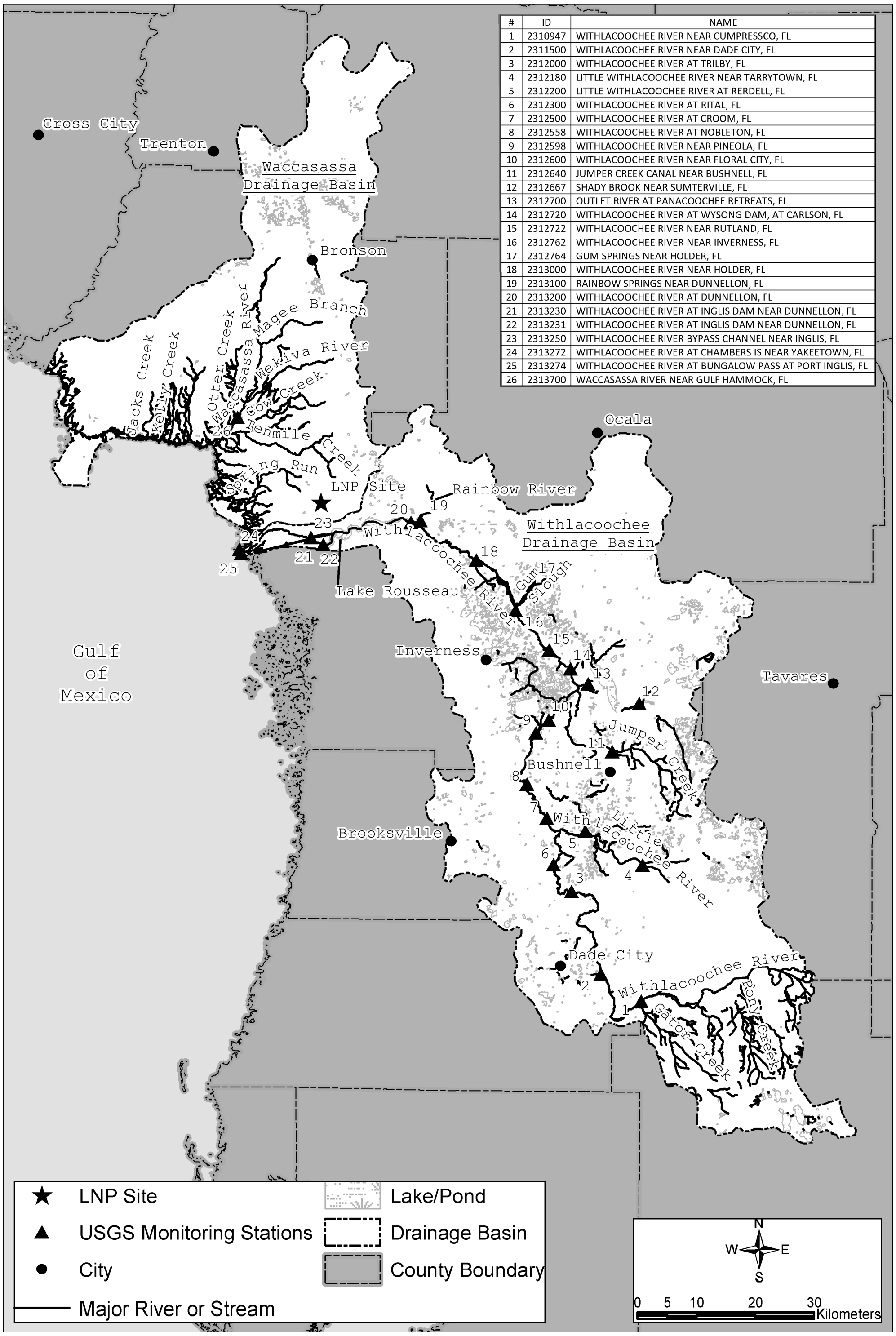


Figure 2.4.1-3. The Withlacoochee and Waccasassa River Basins



#	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

Inglis Dam has a reinforced concrete, two-bay, gated spillway with ogee weirs with a crest elevation of 8.5 m (28 ft) MSL. The maximum allowable lake level is 8.5 m (28 ft) MSL. 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 7,401 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. 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.

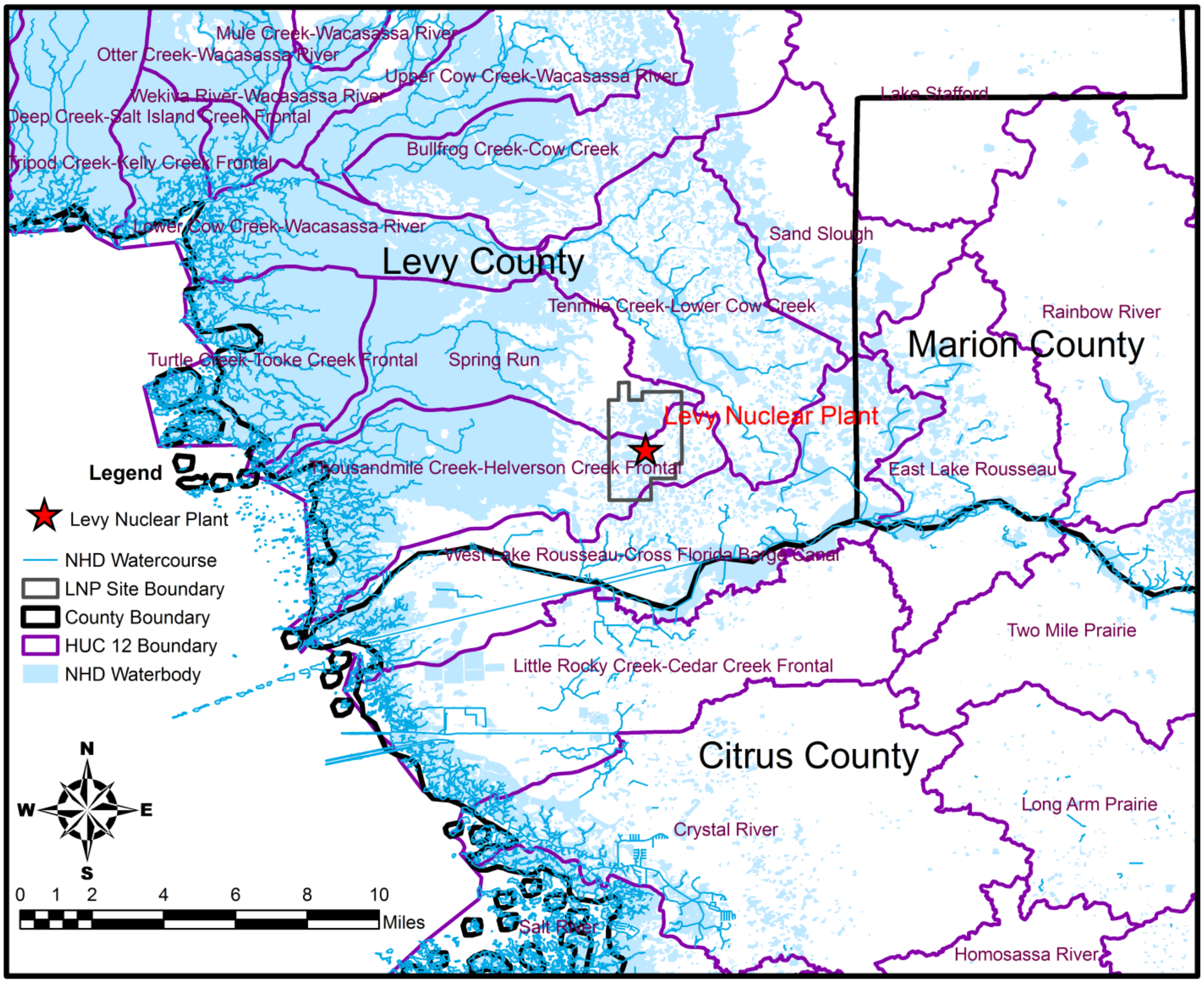


Figure 2.4.1-5. Subwatersheds Near the LNP Site. Waterbodies and watercourses data were obtained from the National Hydrography Dataset.

2.4.1.4 Post-Combined License Activities

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

2.4.1.5 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 to allow the staff to evaluate, as documented in Section 2.4.1, 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. 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 structures, systems, and components (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:

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 17 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

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. 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 2.4.2-4.

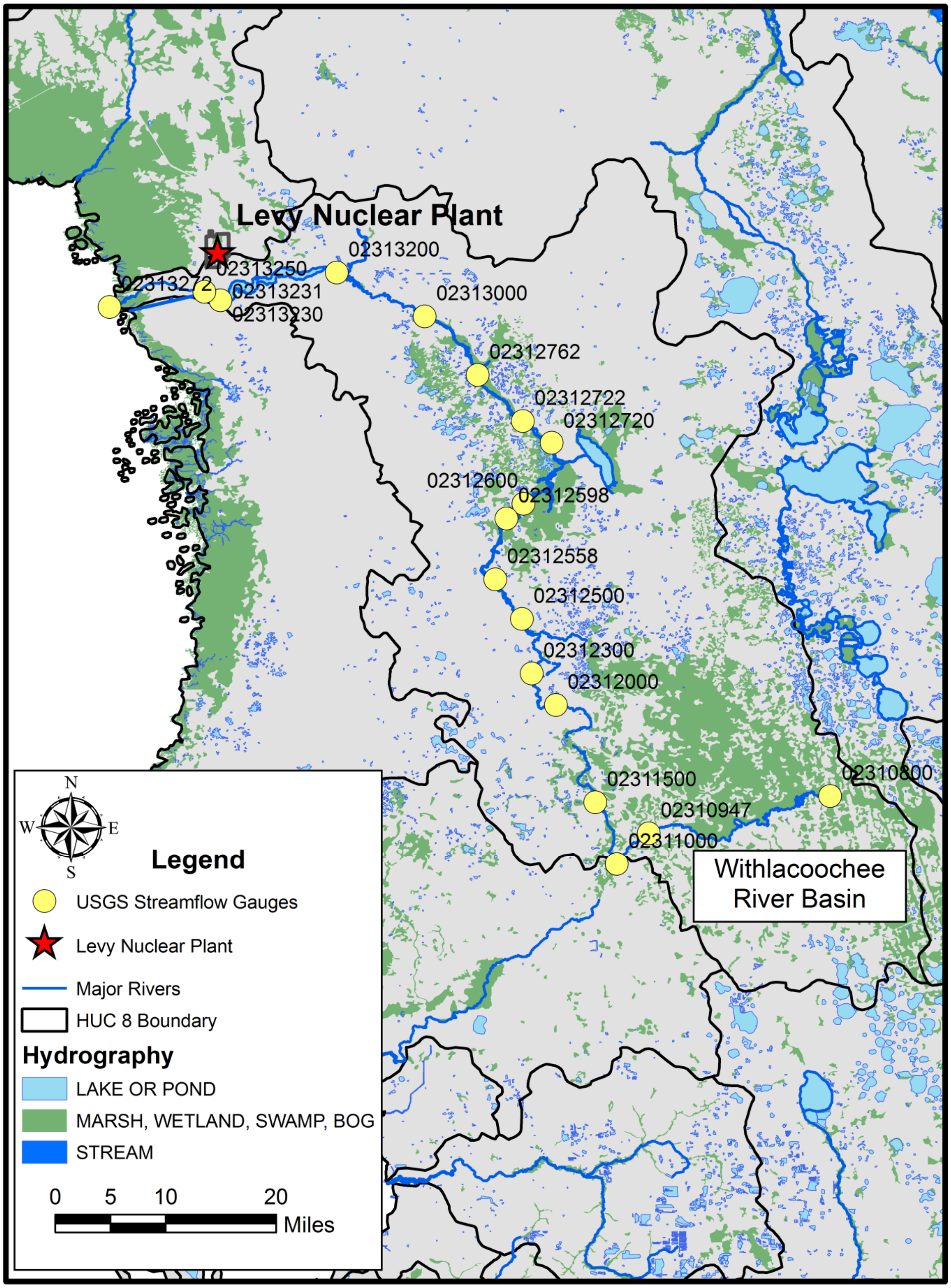


Figure 2.4.2-1. The Withlacoochee River Basin and USGS Streamflow Gauges

Table 2.4.2-4. 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 2.4.2-4. 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.

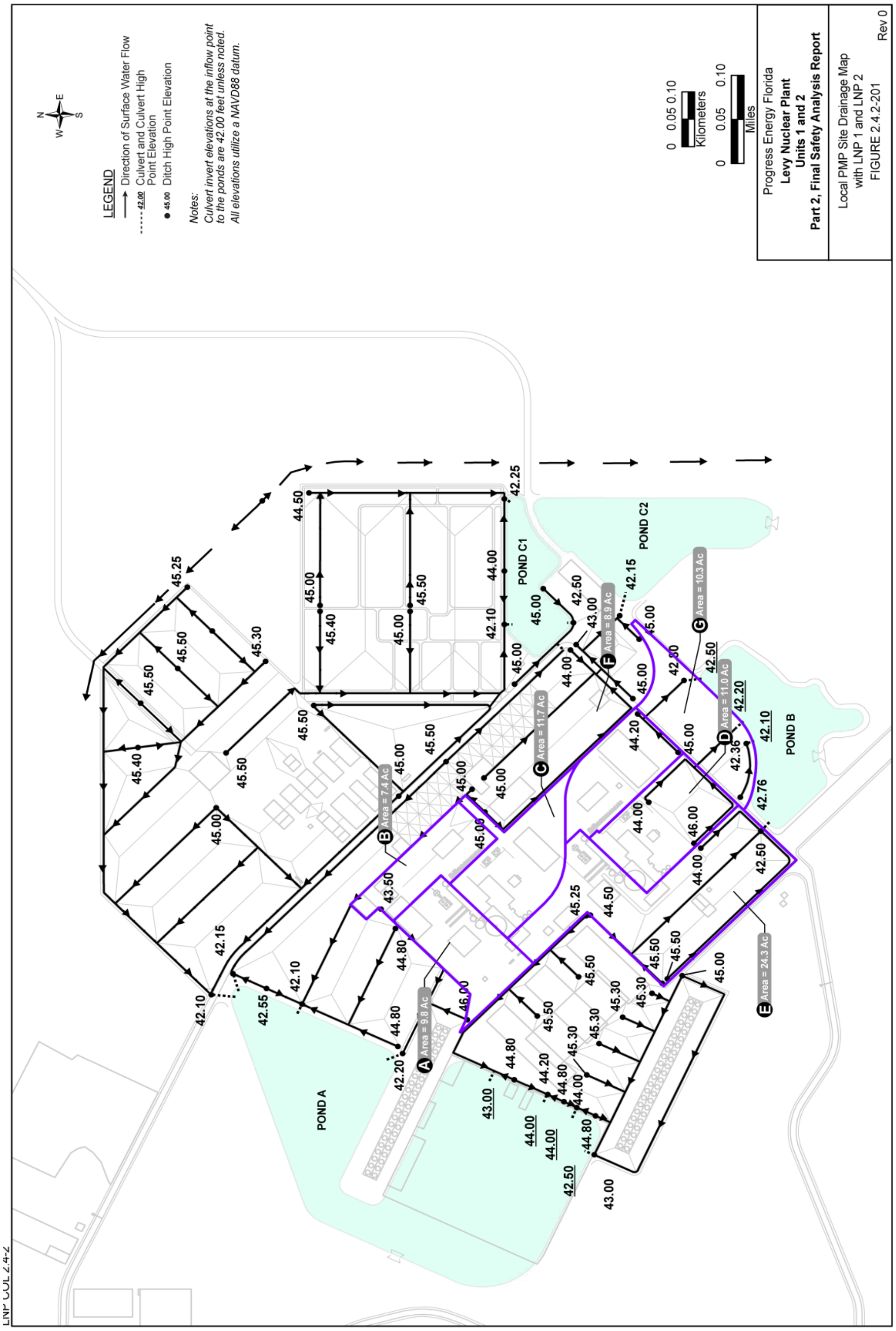


Figure 2.4.2-2. Grading and Site Drainage at the LNP Site

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 2.4.2-2 shows the local PMP values estimated by the applicant.

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 2.4.2-3.

In the FSAR, the applicant stated that roads or railroad tracks 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.

Table 2.4.2-2. 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)

Table 2.4.2-3. 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)

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 below 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	21.0 (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	10.9 (26.9)	32.8 (1160)	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 to allow the staff to evaluate, as documented in Section 2.4.2 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. 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:

AP1000 COL Information Item

- LNP COL 2.4-2

This section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 17 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.

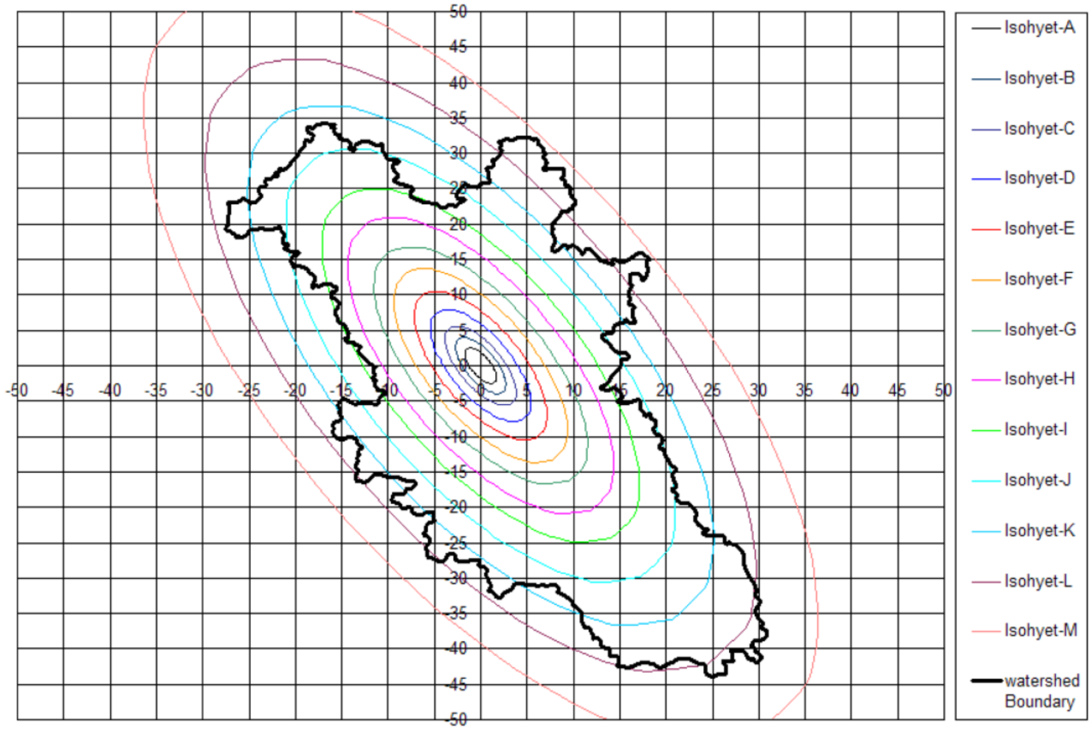
The applicant estimated the values of the isohyets corresponding to the maximum precipitation volume within the Withlacoochee River Basin for the three 6-hour durations with the highest incremental precipitation using the procedure described in HMR 52 (Hansen et al. 1982). The PMP spatial pattern size 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)

2.4.3.3.2 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.

2.4.3.3.3 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 May 7, 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 May 7, 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**.

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 except for the commitments made by the applicant as described in Confirmatory Item 2.4.3-1.

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 to allow the staff to evaluate, as documented in Section 2.4.3, 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. 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 17 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 1,716 m^3/s (60,597 cfs).

The applicant used the USACE HEC-RAS model to simulate a steady flow of 1,699 m^3/s (60,000 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 7.51 m (24.65 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)

Saddle Creek Settling Area No. 4[3]	2,999,828 (2,432)	7.3 (24)	582.8 (20,581.0)
Saddle Creek Settling Area No. 5[3]	12,680,193 (10,280)	16.8 (55)	2,493.3 (88,050.6)
Saddle Creek Settling Area No. 6[3]	62,908 (51)	13.7 (45)	406.3 (14,350.3)
Saddle Creek Settling Area No. 7[3]	12,433,497 (10,080)	4.9 (16)	536.2 (18,935.4)

1. Estimated using Froehlich (1995) approach.
2. Combined Peak Discharge is 3,328.5 m^3/s (117,545.5 cfs)
3. Combined Peak Discharge is 6,523.9 m^3/s (230,388.4 cfs)

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 7.51 m (24.65 ft) NGVD29. The applicant concluded that the LNP site would not be adversely affected by this flood.

LNP COL 2.4-2

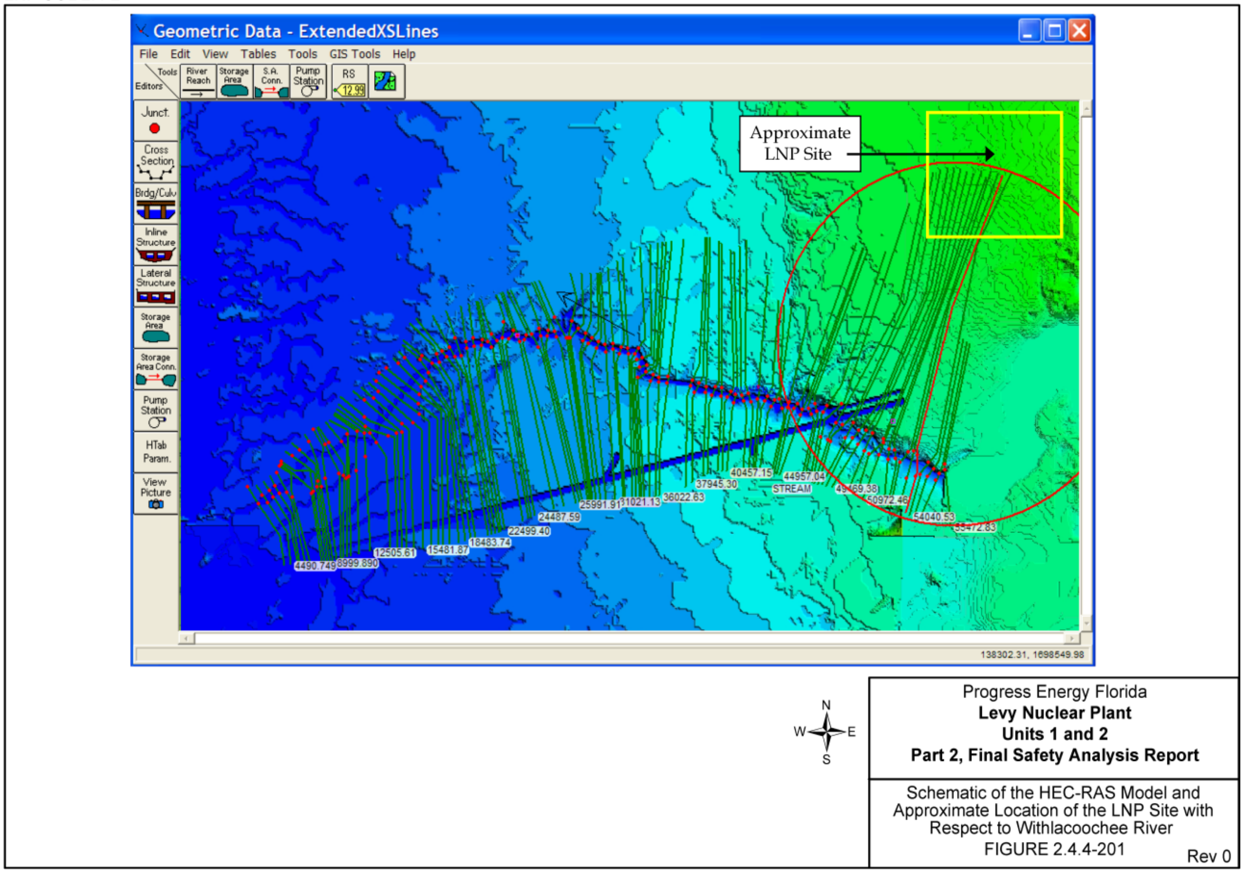

Figure 2.4.4-1. The Cross Sections Used in the HEC-RAS Simulation by the Applicant

<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 7.51 m (24.65 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 to allow the staff to evaluate, as documented in Section 2.4.4, 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. 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:

<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 17 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 2.4.4 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 Rev 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 strom 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)[1] 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

[1] Schwerdt et al., 1979.

(4) maps of highest PMSS water surface elevation at and near the LNP site, and

(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**.

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

Information Submitted by the Applicant

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.

NRC Staff's Technical Evaluation

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

Information Submitted by the Applicant

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.

NRC Staff's Technical Evaluation

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 except for the commitments made by the applicant as described in Confirmatory Item 2.4.5-1.

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 to allow the staff to evaluate, as documented in Section 2.4.5, 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. 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, (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 17 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 tsuami 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 PMTanalysis and text presented in LNP calculation package LNG-0000-X7C-043, Rev. 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 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% exeedance 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 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. RAI 02.04.06-14 issued. The applicant responded to the staff’s RAI 02.04.06-14 in a letter dated March 25, 2010 (ML1009102991) with additional details, information source and FSAR revision. Based on the applicant's detailed response, 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, theyare 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 November30, 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 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 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.
>
> <u>Tsunami Runup</u>: (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 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 □ 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°; latitiude=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 near the Levy site is not evident in this image

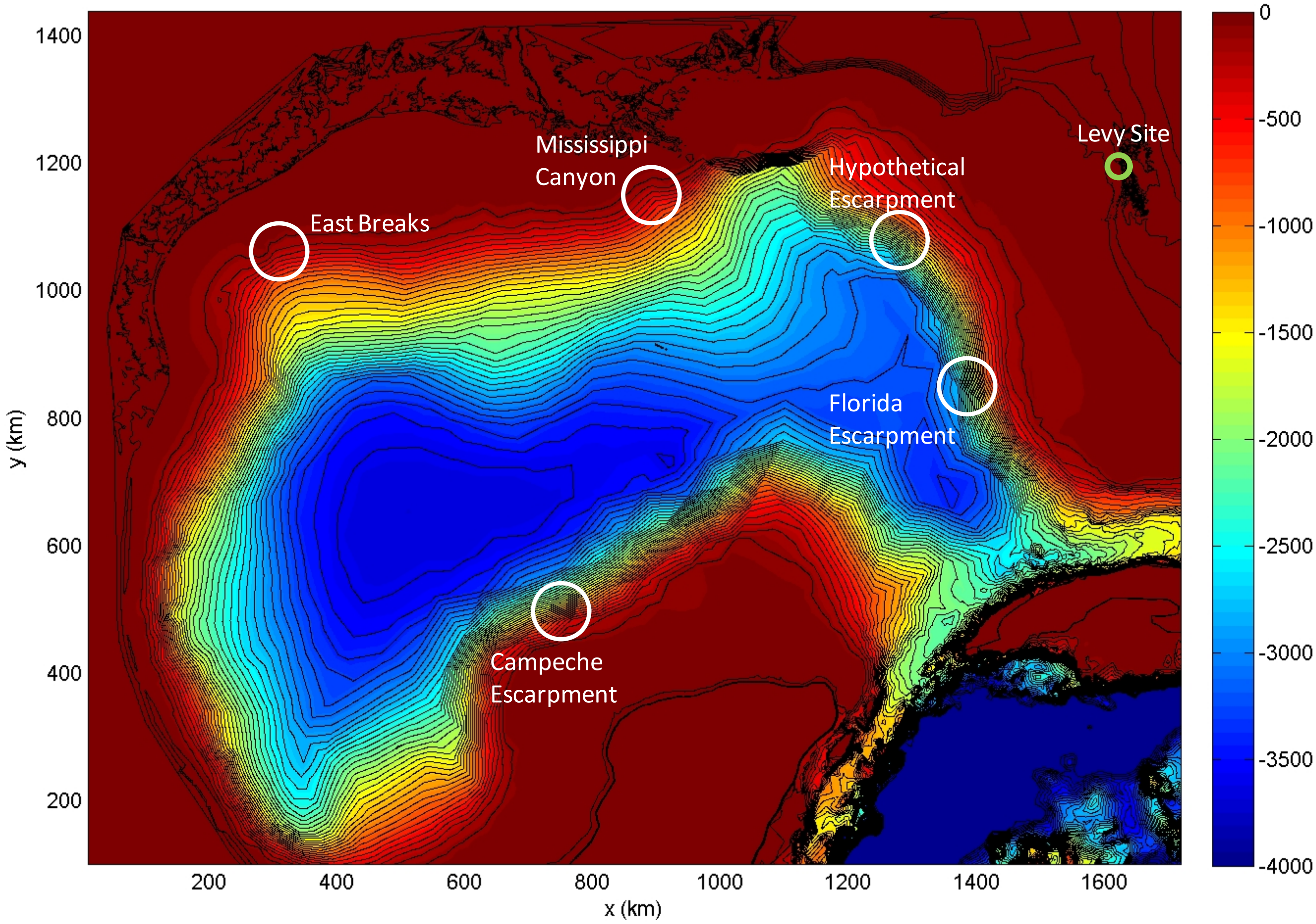

Figure 2.4.6.4.5-1. Bathymetry/topography contour surface of the GOM domain used for the tsunami hydrodynamic modeling. General locations of the five potential tsunami sources are shown by the white circles and the Levy site by the green circle. Bottom elevations are indicated by colors following the colorbar, with units in meters.

Initial Numerical Simulations – Physical Limits

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.

<u>1HD Results (No friction)</u>: The depth transect is taken from the source location directly to the Levy site, as shown in Figure 2.4.6.4.5-8. 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.

1HD Results (Friction): 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.

Campeche Landslide Source:

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).

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. This is most evident in Figure 2.4.6.4.5-20, which shows the tsunami approaching the site. The wave in the nearshore zone is shown in the three lower subplots, which show a zoom-in of the transect near the site. For perspective, the area shown in the lower three plots in these figures is outlined by the red box in the top figure.

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 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.

Florida Slope Landslide Source:

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

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 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.

<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 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>: 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% 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)

1HD Results (No Friction): The depth transect is taken from the source location directly to the Levy site, as shown in Figure 2.4.6.4.5-41. 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-likevegetation 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): 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 as shown in Figure 2.4.6.4.5-49. 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. 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 (Figure 2.4.6.4.5-61), its elevation is approximately 7 m. Note that for the 1HD simulation, the wave height at this depth was 25 m.

Local Evolution of the Tsunami in the Nearshore Areas of the Site

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. Time series of sea surface elevation from these two simulations, at a depth of 30 m, are shown in Figure 2.4.6.4.5-62. Interestingly, the peak elevation 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 given in Figure 2.4.6.4.5-62. Figures 2.4.6.4.5-63 through 2.4.6.4.5-67 are plan-view snapshots of the wave train attacking the coastline. 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% 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% exeedance 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 hydrodgraphy 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.

2.4.6.5 *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 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 and 10 CFR Part 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 17 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 to allow the staff to evaluate, as documented in Section 2.4.7, 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. 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*

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 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.

2.4.9.4.1 Post-Combined License Activities

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

2.4.9.5 *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 Floods

In addition, this section addresses the following COL-specific information identified in Section 2.4.1.2 of Revision 17 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 2.4.16 of Revision 17 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 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 to allow the staff to evaluate, as documented in Section 2.4.10, 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.

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 17 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 Si Kay – keep together "safety-related"

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 thevicinity 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 and that there are no site characteristics in the DCD associated with low water conditions.

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 to allow the staff to evaluate, as documented in Section 2.4.11, 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. This addresses COL information item 2.4-3.

2.4.12 Groundwater

2.4.12.1 *Introduction*

FSAR Rev 2 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 17 of the DCD, and is defined and discussed in Section 2.4.1.4 of Revision 17 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 120 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 (494 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 Rev 2.

2.4.12.4.2 Groundwater Sources, Present and Future Groundwater Use

Information Submitted by the Applicant

The applicant determined that within 40 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-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 km (10 mi) of the LNP site. The numbers and types of permitted wells were tabulated by Township Range and Section in FSAR Rev 2. 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 2 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 (1.27 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 38.1 m (122 to 125 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 (ML092150960). 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 RAI 02.04.12-08 Figure 1. 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 (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 0.36.6 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 Rev. 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 Rev. 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 Rev 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.

However, 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 2.4.12-14 and determined that it would be appropriate to use a porous media conceptual model for the groundwater velocity (seepage 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. 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 reasonable 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 Rev 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 MLd (21.96 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 36 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 Rev 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 this Section of LNP FSAR Rev 2, the applicant refers to the previous section 2.4.12.4.2, titled "Groundwater Sources," and to Section 2.4.13 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 Rev 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 Rev 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 preapplication 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 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 Rev 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 SCCs during rainfall events. The applicant described planned grading of the site to drain surface flow away from the safety-related SCCs. 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**.

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 except for the commitments made by the applicant as described in Confirmatory Item 2.4.12-1. 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 to allow the staff to evaluate, as documented in Section 2.4.12, 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. This addresses COL information item 2.4-4. In conclusion, the applicant has provided sufficient information for satisfying 10 CFR Part 52 and 10 CFR Part 100.

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 17 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 Dispersions 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 105,988 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 Rev 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 1.9 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 Rev 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.

Seepage velocities used in the calculation were presented in Section 2.4.12 of LNP FSAR Rev 2. 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. The evaluation presented in FSAR Rev. 1 assumed longitudinal and transverse dispersivities of $\alpha L = 1$ m and $\alpha L^*\alpha T = 1$ m^2, respectively. The dispersion coefficients for x directed flow are $Dx=\alpha L^*Ux$ and $Dy=Dz=\alpha T^*Ux$ where Ux is the seepage velocity in the direction of groundwater flow. FSAR Rev. 1 references NUREG/CR-3332 (EPA 1983) to show that longitudinal dispersivity of $\alpha L = 10$ to 15 m (32.8 to 49.2 ft) for limestone and carbonate aquifers are reasonable. Lower dispersivity values used in the analysis will result in higher concentrations of radionuclides at the receptor locations.

The LNP FSAR Rev 2 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 Rev 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 conc. – tritium = 5.2E-04 pCi/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 Rev 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 Rev 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 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 Rev 2 analysis.

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.

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 Rev. 1) on the total dose calculated at the hypothetical downgradient well.

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 Rev 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 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 Rev 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 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 Rev 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.

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.

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 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 Rev 2 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.

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, as discussed in the following paragraphs, the seepage velocity values used in the transport calculations were found to not be conservative in the analysis presented in LNP FSAR Rev 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 expected 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, to allow the staff to evaluate, as documented in this section, 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, and with respect to 10 CFR 20 as it relates to effluent concentration limits. This addresses COL information item 2.4-5. In conclusion, the applicant provided sufficient information for satisfying 10 CFR Part 20, 10 CFR Part 52, and 10 CFR Part 100.

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 2.4.16 of Revision 17 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 to allow the staff to evaluate, as documented in Section 2.4.14 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. This addresses COL Information Item 2.4-6.