Progress Energy

Serial: NPD-NRC-2011-052
June 21, 2011

10 CFR 52.79

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

LEVY NUCLEAR PLANT, UNITS 1 AND 2
DOCKET NOS. 52-029 AND 52-030

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION LETTER NO.104 RELATED TO PROBABLE MAXIMUM SURGE AND SEICH FLOODING

Reference: Letter from Brian C. Anderson (NRC) to John Elnitsky (PEF), dated May 19, 2011, "Request for Additional Information Letter No. 104 Related to SRP Section 2.4.5 for the Levy County Nuclear Plant, Units 1 and 2 Combined License Application"

Ladies and Gentlemen:

Progress Energy Florida, Inc. (PEF) hereby submits our response to the Nuclear Regulatory Commission's (NRC) request for additional information provided in the referenced letter.

A response to the NRC request is addressed in the enclosure. The enclosure also identifies changes that will be made in a future revision of the Levy Nuclear Plant Units 1 and 2 application.

If you have any further questions, or need additional information, please contact Bob Kitchen at (919) 546-6992, or me at (727) 820-4481.

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

Executed on June 21, 2011.

Sincerely,

John Elnitsky
Vice President
New Generation Programs & Projects

Enclosure

cc : U.S. NRC Region II, Regional Administrator
Mr. Brian C. Anderson, U.S. NRC Project Manager

Progress Energy Florida, Inc.
P.O. Box 14042
St. Petersburg, FL 33733

D094
NRO

Levy Nuclear Plant Units 1 and 2
Response to NRC Request for Additional Information Letter No. 104 Related to SRP Section 2.4.5 for the Combined License Application, dated May 19, 2011

NRC RAI #	Progress Energy RAI #	Progress Energy Response
02.04.05-11	L-0937	Response enclosed – see following pages

NRC Letter No.: LNP-RAI-LTR-104

NRC Letter Date: May 19, 2011

NRC Review of Final Safety Analysis Report

NRC RAI NUMBER: 02.04.05-11

Text of NRC RAI:

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 National Oceanic and Atmospheric Administration (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 plant, (3) provide estimates of coincident wind wave runup, (4) maps of highest probable maximum storm surge (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 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 non-linear 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.

PGN RAI ID #: L-0937

PGN Response to NRC RAI:

Progress Energy's response to NRCs comments is provided as follows:

A confirmatory analysis for PMH surge water level at the LNP site using the Sea, Lake, and Overland Surges from Hurricanes (SLOSH) computer model and associated COL application revisions were presented in the response to NRC RAI 2.4.5-10 (NPD-NRC-2011-004, L-0876). In that confirmatory analysis the initial water level was estimated by adding the long-term sea level rise to the 10 percent exceedance high tide based on observed spring high tide data for the region for the period 1983 to 2010.

As requested in this RAI the following items are provided:

(1) b An updated Probable Maximum Storm Surge (PMSS) maximum water surface elevation at the LNP site is estimated with combination of (i) maximum stillwater elevation for a SLOSH simulation carried out with an initial water surface elevation estimated following the guidelines of Regulatory Guideline 1.59 (RG 1.59) (Reference 2.4.5-11-1) and using more recent tide data and (ii) wind-wave effects using the Coastal Engineering Manual (CEM) (References 2.4.5-11-8 and 2.4.5-11-9) approach.

(2) An update to FSAR text that describes how the CEM approach is used to estimate wind-wave activity coincident with PMSS maximum water surface elevation at the LNP site.

(3) Updates to FSAR that describe appropriately selected PMSS characteristics at the LNP site and a discussion of available margins between the DCD Maximum Flood Level site parameter and the highest PMSS water surface elevation including coincident wind-wave activity.

PMSS Maximum Water Surface Elevation:

10 Percent Exceedance High Tide Level:

The maximum water level due to Probable Maximum Hurricane (PMH) at the Levy Nuclear Plant (LNP) site was presented as Scenario 1 in FSAR Subsection 2.4.5.4. The initial water level for Scenario 1 was estimated by adding the long-term sea level rise to the 10 percent exceedance high tide based on observed spring high tide data for the period 1983 to 2010. As the 10 percent exceedance high tide was computed based on the observed spring high tide values, the initial rise was not applicable and therefore was not added to obtain the 10 percent exceedance high tide level.

As requested in the RAI, Item (1)b, following RG 1.59, the 10 percent exceedance high tide is estimated using the following approaches:

- **Approach A:**

The 10 percent exceedance spring high tide for Crystal River is determined directly from Table C.1 in RG 1.59. This value specified with respect to Mean Low Water (MLW) is converted to NAVD88.

The 10 percent exceedance spring high tide from Table C1 is obtained as 4.3 ft. MLW. Two different sources were used to convert the 10 percent exceedance spring high tide from MLW to NAVD88. The first source was the NOAA Tides and Currents Online site for the Cedar Key tide gage (Reference 2.4.5-11-4) and the second source was the National Geodetic Survey (Reference 2.4.5-11-5) which is also cited in Reference 2.4.5 11 4. Using the datum conversion chart in Reference 2.4.5-11-4 resulted in a 10 percent exceedance spring high tide of 2.68 ft. NAVD88 and using the datum conversion chart in Reference 2.4.5-11-5 resulted in a 10 percent

exceedance spring high tide of 2.66 ft. NAVD88. Therefore, the higher value of 2.68 ft. NAVD88 was conservatively considered as 10 percent exceedance spring high tide in this approach.

Approach B:

The harmonic astronomical tidal constituents at Cedar Key for the latest tidal epoch were obtained from Reference 2.4.5-11-2. The latest tidal epoch, according to Reference 2.4.5-11-3, is from 1983 to 2001 and consequently the harmonic astronomical tidal constituents obtained from Reference 2.4.5-11-2 for this epoch are the most recent set for Cedar Key. The aforementioned tidal constituents were used to compute the predicted maximum monthly astronomical tide values over the 21-year period. The 10 percent exceedance spring high tide, as outlined in RG 1.59, was obtained from the predicted maximum monthly astronomical tide values. Using the above method the 10 percent exceedance high tide is obtained as 2.63 ft. NAVD88.

The 10 percent exceedance spring high tides obtained using Approach A and Approach B are shown below.

Approach	10 percent Exceedance Spring High Tide (ft. NAVD88)
A	2.68
B	2.63

The higher value of 2.68 ft. NAVD88 obtained from Approach A is conservatively used for further analysis.

Initial Water Level:

The initial water level is obtained by adding the initial rise of 0.6 ft. (Table C1 of RG 1.59; Reference 2.4.5-11-1) and long-term sea level rise of 0.59 ft. discussed in FSAR Subsection 2.4.5.4.4 to the 10 percent exceedance predicted high tide computed above. Therefore the initial water level is 3.87 ft. NAVD88 (2.68' + 0.6' + 0.59'). This initial water level of 3.87 ft. NAVD88 is higher than the initial water level of 3.82 ft. NAVD88 discussed for Scenario 1 in FSAR Subsection 2.4.5.4.4. Therefore, an initial water level of 3.87 ft. NAVD88 is conservatively used in the SLOSH model run for this analysis.

SLOSH Model Run:

Using the higher initial water level of 3.87 ft. (NAVD88), and all other input data used for Scenario 1 discussed in FSAR Subsection 2.4.5.4, an additional SLOSH model run was performed. Figure RAI 2.4.5-11-1 provides the SLOSH display screenshot at the time of the peak PMH surge level.

As shown in Figure RAI 2.4.5-11-1, the PMH maximum surge elevation from the SLOSH model run is 47.7 ft. NAVD88 which is the same as the maximum surge elevation computed for Scenario 1.

Wind-wave Runup Using Coastal Engineering Manual (CEM) Approach:

The maximum water level near the LNP site due to the PMH is estimated as 47.7 ft. NAVD88. A water level of 48.3 ft. NAVD88 including 0.6 ft. of wave setup is used for the wave runup calculation as discussed in FSAR Subsection 2.4.5.4.8.

The Automated Coastal Engineering System (ACES) application of the Coastal Engineering Design and Analysis System (CEDAS) software, distributed by Veri-Tech was used, to compute the design waves and the wave runup at the LNP site (References 2.4.5-11-6 and 2.4.5-11-7). CEDAS is a software suite developed by the U.S. Army Corps of Engineers (USACE) to automate the equations provided in the Coastal Engineering Manual (CEM) (References 2.4.5-11-7, 2.4.5-11-8, and 2.4.5-11-9). In most instances, CEDAS follows the exact equations as provided in the CEM; however, there are instances where the specific algorithm used for computation in CEDAS differs from the equations provided in the CEM.

For calculations of wave runup on embankments and structures, ACES uses the equations from Ahrens and Titus (1985) (Reference 2.4.5-11-10) to determine runup of regular waves, whereas the CEM uses a method that can be adjusted for random waves developed by de Waal and van der Meer (1992) (Reference 2.4.5-11-11), which computes the runup exceeded by 2 percent of the random waves. The depth of water at the embankment slope is very shallow and due to the high hurricane winds the waves will break. Therefore, the design wave height at the toe of the fronting embankment is strictly determined by the water depth and the regular wave methodology of Ahrens and Titus (Reference 2.4.5-11-10) is actually more appropriate. Nevertheless, if proper reduction factors in the de Waal and van der Meer equation (Reference 2.4.5-11-11) are selected, results identical to those calculated by the Ahrens and Titus method are obtained (Reference 2.4.5-11-10). The following provides the wave runup computation using the two methodologies.

Wave Run-up Using ACES/CEDAS

The maximum PMH surge water level including wind-wave setup = 48.3 ft. NAVD88,

Grade elevation at toe of structure = 47.0 ft. NAVD88,

Design water depth at the toe of the power plant embankment = 48.3 – 47.0 = 1.3 ft.

Because of this shallow depth of water at the embankment slope and due to the high hurricane winds the waves will break.

From CEM Equation II-2-39 (Reference 2.4.5-11-8), a limiting wave period of TP = 1.96 seconds is computed based on this water depth. Also, adopting the standard breaking wave criteria of Hs = $0.78*d_s$, the maximum wave height that could propagate to the structure without breaking is 1.0 foot. The structure is assumed to be impermeable with a slope of 3H:1V.

Using these design parameters into ACES yields a 1.481 ft. maximum wave runup on the structure. The ACES/CEDAS output for wave runup on the embankment slope is provided in Figure 1.

Figure 1. ACES/CEDAS Output for Wave Runup on Impermeable Embankment Slope

Wave Runup and Overtopping on Impermeable Structures

Wave type: Monochromatic Slope type: Smooth
Rate estimate: Runup and Overtopping

Breaking criteria:	0.780				
Incident wave ht (Hi):	1.000	ft	Wave Runup (R):	1.481	ft
Peak wave period (T):	1.960		Onshore wind velocity (U):	227.330	ft/sec
COTAN of nearshore slope (cot phi):	1429.000		Deepwater wave (Ho):	1.021	ft
Water depth at structure toe (ds):	1.300	ft	Relative height (ds/Ho):	1.273	
COTAN of structure slope (cot theta):	3.000		Wave steepness (Ho/gT²):	0.008	
Structure height above toe (hs):	3.000	ft	Overtopping coef(alpha):	0.076	
			Overtopping coef(Q*o):	0.045	
			Overtopping rate (Q):	0.000	ft³/s-ft

Wave Run-up Using CEM Equations

CEM Equation VI-5-7 (Reference 2.4.5-11-9) for runup of (random) waves on impermeable slopes (de Waal and van der Meer, 1992; Reference 2.4.5-11-11) is given by:

$$\frac{R_{u2\%}}{H_s} = 1.5\xi_{op}\gamma_r\gamma_b\gamma_h\gamma_\beta \qquad \text{Equation 2.4.5-11-1}$$

Where:

$R_{u2\%}$ is the runup level exceeded by 2 percent of the incident waves,
H_s is the significant wave height,
ξ_{op} is the surf similarity parameter,
γ_r is the reduction factor for surface roughness (γ_r = 1 for smooth slopes),
γ_b is the reduction factor for influence of a berm (γ_b = 1 for non-bermed profiles),
γ_h is the reduction factor for non-Rayleigh distributed waves (γ_h = 1 for Rayleigh distribution),
γ_β is the reduction factor for angle of incidence, β, of the waves (γ_β = 1 for head-on waves).

This equation includes reduction factors (coefficients) that are selected based upon the specifics of the wave environment and character of the structure. They are applied to account for the influence of surface roughness, γ_r, the presence of a berm γ_b, shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution, γ_h, (as is the case herein), and the angle of incidence of the waves, γ_β.

The surf similarity parameter is defined by CEM Equation VI-5-2 (Reference 2.4.5-11-9) as,

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{gT_p^{\,2}}}} \qquad \text{Equation 2.4.5-11-2}$$

Where:

α is the slope angle,
g is the acceleration due to gravity (32.2 ft/s^2),
H_s is the significant wave height, and
T_p is the peak wave period.

Using the wave parameters determined above

tan α = 1/3 = 0.3333

g = 32.2 ft/s^2,
H_s = 0.78 x water depth of 1.3 = 1.0 foot
T_p = 1.96 sec.

the surf similarity parameter, ξ_{op}, is 1.479.

Because the design wave is a regular, depth-limited wave, and is not based on a Rayleigh distribution, a reduction factor γ_h is calculated using CEM Equation VI-5-10 (Reference 2.4.5-11-9).

$$\gamma_h = \frac{H_{2\%}}{1.4H_s} \qquad \text{Equation 2.4.5-11-3}$$

Assuming that all waves are uniform under the depth-limited condition ($H_{2\%} = H_s$),

$$\gamma_h = \frac{1}{1.4} = 0.71$$

A surface roughness reduction factor, γ_r, of 0.94 is used based on the condition that the embankment slope is covered with grass. Table VI-5-3 (page VI-5-11 of Reference 2.4.5-11-9) provides a range for γ_r and 0.94 was chosen as the appropriate value for this application.

With no berm (γ_b = 1), normally incident waves (γ_β = 1), and a significant wave height of 1.0 foot (H_s = 1.0).

Substituting the above values in the Equation 2.4.5-11-1

Runup $R_{u2\%} = H_s$ x (1.5 x 1.479 x 0.94 x 1 x 0.71 x 1) = 1.481 ft.

The calculated wave run-up height of 1.481 ft. is same as the run-up calculated by the ACES/CEDAS software described above.

Maximum Water Level Including Wave Run-up at LNP Site

Table 1 presents a summary of the total PMSS at the site with consideration from the maximum PMH surge value at the site as well as contributions from wave setup and wave runup.

Table 1. Total PMH Surge Elevation at the LNP Site Including Wave Effects

Component	Units	Value
LNP Site Grade Elevation	FEET NAVD88	**50.00**
10 percent Exceedance Spring High Tide Elevation Including the Initial Rise.	FEET NAVD88	**3.28**
Long-term Sea Level Rise	FEET	**0.59**
Initial Water Level	FEET NAVD88	**3.87**
SLOSH Surge Elevation with Initial Water Level Including 10 percent Exceedance High Tide, Initial Rise and Long-term Sea Level Rise	FEET NAVD88	**47.70**
Wave Setup	FEET	**0.60**
Wave Runup	FEET	**1.48**
TOTAL PMSS Including Wave Effects	FEET NAVD88	**49.78**

As requested in the RAI, Item (2), the updates to the FSAR based on above discussion of CEM approach to evaluate wind-wave activity coincident with PMSS maximum water surface elevation at the LNP site are provided in the "Associated LNP COL Application Revisions."

The Available Margin Between the DCD Maximum Flood Level and Maximum PMH Surge Level Including Wave Run-up at the LNP Site:

As requested in the RAI, Item (3), the available margins are discussed in the following paragraph.

As shown in FSAR Table 2.0-201, the DCD plant elevation for the LNP site is 100 ft. which is equivalent to 51 ft. NAVD88. The PMH maximum water surface elevation including wind-wave effects is 49.78 ft. NAVD88. The highest PMH water surface elevation of 49.78 ft. NAVD88 provides 1.22 ft. of margin to the DCD Maximum Flood Level site parameter of 51.0 ft. NAVD88.

The makeup water pump house intake is located on the Cross Florida Barge Canal (CFBC). The pump house provides cooling water for normal operation of LNP 1 & 2 and is not safety-related. Therefore, low water level in the CFBC due to PMH will not affect the safety functions of LNP 1 & 2. A discussion of low water considerations is presented in FSAR Subsection 2.4.11.

References:

2.4.5-11-1 U.S. Nuclear Regulatory Commission, Regulatory Guide 1.59 Design Basis Floods for Nuclear Power Plants, Revision 2, August 1977.

2.4.5-11-2 National Oceanic and Atmospheric Administration (NOAA)'s web site for tides and currents Link: http://tidesandcurrents.noaa.gov/geo.shtml?location=8727520; for Cedar Key, Florida, station 8727520, Accessed on June 26, 2011.

2.4.5-11-3 National Oceanic and Atmospheric Administration (NOAA)'s web site for tides and currents Link: http://tidesandcurrents.noaa.gov/datum_options.html, Latest tidal epoch from 1983 to 2001 for Cedar Key, Accessed on June 26, 2011.

2.4.5-11-4 National Oceanic and Atmospheric Administration (NOAA)'s web site for tides and currents online datum conversion chart , Link:http://tidesandcurrents.noaa.gov/data_menu.shtml?stn=8727520%20Cedar%20Key,%20FL&type=Datums, Used for Cedar Key, Florida, Accessed on June 26, 2011.

2.4.5-11-5 National Oceanic and Atmospheric Administration (NOAA)'s web site for National Geodetic Service datum conversion chart: http://www.ngs.noaa.gov/newsys-cgi-bin/ngs_opsd.prl?PID=AR1204&EPOCH=1983-2001, Used for datum conversion, Accessed on June 26, 2011.

2.4.5-11-6 U.S. Army Corps of Engineers, Automated Coastal Engineering System Technical Reference, Part 5 Chapter 2, September 1992.

2.4.5-11-7 U.S. Army Corps of Engineers, Coastal and Hydraulics Laboratory, Automated Coastal Engineering System (ACES) Coastal Engineering Design and Analysis System (CEDAS) Web link: http://chl.erdc.usace.army.mil/cedas.

2.4.5-11-8 U.S. Army Corps of Engineers, Coastal Engineering Manual EM 1110-2-1100. Part II Chapter 2, June 2006.

2.4.5-11-9 U.S. Army Corps of Engineers, Coastal Engineering Manual EM 1110-2-1100. Part VI Chapter 5, June 2006.

2.4.5-11-10 Ahrens, J.P., and Titus, M.F. 1985. "Wave Runup Formulas for Smooth Slopes," Journal of Waterway, Port, Coastal and Ocean Engineering, American Society of Civil Engineers, Vol. 111, No. 1, pp. 128-133.

2.4.5-11-11 de Waal, J.P., and van der Meer, J.W. 1992. "Wave Run-Up and Overtopping on Coastal Structures," Proceedings of the 23rd International Coastal Engineering Conference, American Society of Engineers, Vol. 2, pp. 1758-1771.

Associated LNP COL Application Revisions:

The following changes will be made in a future revision to LNP COLA Part 2, FSAR including updates provided in response to NRC Letter 095 RAI # 02.04.05-10, (NPD-NRC-2011-004, L-0876):

1. Table 2.4.5-223 (As added in response to NRC Letter 095, RAI 02.04.05-10) will be revised per Attachment 02.04.05-11-A.

2. A new Figure 2.4.5-247 will be added per Attachment 02.04.05-11-B.

3. No existing FSAR figures in Subsection 2.4.5 are revised or deleted.

4. A new Subsection 2.4.5.4.7.1 will be added to Subsection 2.4.5.4.7 (As added in response to NRC Letter 095, RAI 02.04.05-10) as follows:

   2.4.5.4.7.1 Additional SLOSH Run with Higher Initial Water Level Based on RG 1.59

   In Subsection 2.4.5.4.4 the 10 percent exceedance spring high tide was computed based on observed tide data. The 10 percent exceedance spring high tide was also determined following the Regulatory Guide 1.59 (RG 1.59). The 10 percent exceedance spring high tide value of 4.3 ft. MLW for Crystal River was directly obtained from Table C1 of RG 1.59. As this value from RG 1.59 is based on MLW datum, it was converted to NAVD88 using the datum conversion chart from Reference 2.4.5-226. Using the above conversion, the 10 percent exceedance high tide value of 4.3 ft. MLW is converted to 0.82 m (2.68 ft.) NAVD88. Also using the latest tidal epoch from Reference 2.4.5-227 for the period of 1983 to 2001 for Cedar Key, and following the RG 1.59, the 10 percent exceedance, predicted high tide level is computed to be 0.80 m (2.63 ft.) NAVD88. Conservatively, the 10 percent exceedance predicted high tide value of 0.82 m (2.68 ft.) NAVD88, computed using Table C1 of RG 1.59, was combined with initial rise of 0.18 m (0.6 foot) to obtain 10 percent exceedance high tide level of 1.00 m (3.28 ft.) NAVD88. This 10 percent exceedance high tide level of 3.28 ft. NAVD88 is 0.02 m (0.05 foot) higher than 10 percent exceedance high tide level of 3.23 ft. NAVD88 computed using observed tide data. By adding 10 percent exceedance high tide of 3.28 ft. NAVD88 to the long-term sea level rise of 0.59 ft., an initial water level of 1.18 m (3.87 ft.) NAVD88 was obtained.

   In Subsection 2.4.5.4.7 Scenario 1, which resulted in maximum PMH surge, an initial water level of 1.16 m (3.82 ft.) NAVD88 was used based on observed tide data. An additional SLOSH run was performed with a higher initial water level of 3.87 ft. NAVD88 along with other input data used for Scenario 1. The PMH surge level obtained for this run was 14.54 m (47.7 ft.) NAVD88, which is the same as obtained for Scenario 1. Figure 2.4.5-247 shows the maximum surge at the LNP site for this additional SLOSH run.

5. In the third sentence of the third paragraph of Subsection 2.4.5.4.8 (As added in response to NRC Letter 095, RAI 02.04.05-10) revise the text

   "(PMH surge + Wave setup) of 14.7 m (48.31 ft.)"

   to read:

   "(PMH surge + Wave setup) of 14.7 m (48.3 ft.)"

6. The last paragraph of Subsection 2.4.5.4.8 (As added in response to NRC Letter 095, RAI 02.04.05-10) will be revised from:

"The wave runup at the plant.....surge elevation occurs"

to read:

The depth of water at the embankment slope is shallow and the high hurricane winds are expected to generate waves that will break. Therefore, wave runup at the plant buildings was estimated due to a breaking wave generated based on the local depth from the maximum surge elevation of 14.72 m (48.3 ft.) NAVD88. The Coastal Engineering Manual approach was used to estimate the wave runup at the LNP site. Wave runup was computed using the CEM Equation VI-5-7 (Reference 2.4.5-228) which is given by:

$$\frac{R_{u2\%}}{H_s} = 1.5\xi_{op}\gamma_r\gamma_b\gamma_h\gamma_\beta \qquad \text{Equation 2.4.5-6}$$

Where $R_{u2\%}$ is the runup level exceeded by 2 percent of the incident waves,

H_s is the significant wave height,
ξ_{op} is the surf similarity parameter,
γ_r is the reduction factor for surface roughness (γ_r = 1 for smooth slopes),
γ_b is the reduction factor for influence of a berm (γ_b = 1 for non-bermed profiles),
γ_h is the reduction factor for non-Rayleigh distributed waves (γ_h = 1 for Rayleigh distribution), γ_β is the reduction factor for angle of incidence, β, of the waves (γ_β = 1 for head-on waves).

This equation includes reduction factors (coefficients) that are selected based upon the specifics of the wave environment and the character of the structure. They are applied to account for the influence of surface roughness, γ_r, the presence of a berm γ_b, shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution, γ_h, (as is the case herein), and the angle of incidence of the waves, γ_β.

The surf similarity parameter is defined by CEM Equation VI-5-2 (Reference 2.4.5-228) as,

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{gT_p^{\,2}}}} \qquad \text{Equation 2.4.5-11-7}$$

Where: α is the slope angle, g is the acceleration due to gravity (32.2 ft/s^2), H_s is the significant wave height, and T_p is the peak wave period.

The limiting wave period from CEM Equation II-2-39 (Reference 2.4.5-229) is calculated as 1.96 seconds. Using the embankment slope of 3H:1V (tan α = 0.3333) and breaking wave height of 0.78 times depth of water (0.78 x 1.3 ft.), the surf similarity parameter, ξ_{op}, is estimated as 1.479.

Because the design wave is a regular, depth-limited wave, and not based on a Rayleigh distribution, a reduction factor γ_h is calculated using CEM Equation VI-5-10 (Reference 2.4.5-228).

$$\gamma_h = \frac{H_{2\%}}{1.4H_s} \qquad \text{Equation 2.4.5-8}$$

As all waves are uniform under the depth-limited condition ($H_{2\%} = H_s$), a reduction factor γ_h is estimated as 0.71. A surface roughness reduction factor, γ_r, of 0.94 is used based on the condition that the embankment slope is covered with grass. Table VI-5-3 (page VI-5-11 of Reference 2.4.5-228) provides a range for γ_r and 0.94 was chosen as the appropriate value for the LNP site. Conservatively using $\gamma_b = 1$, $\gamma_\beta = 1$ (for normally incident waves), and a significant wave height H_s of 1.0 foot, the wave runup, $R_{u2\%}$ from Equation 2.4.5-6 is calculated to be 0.45 m (1.48 ft.).

7. Add the following sentence after the first sentence in the first paragraph of Subsection 2.4.5.4.9 (As added in response to NRC Letter 095, RAI 02.04.05-10):

   "This table also includes the results from PMH surge with higher initial water level of 1.18 m (3.87 ft.) NAVD88 based on RG 1.59."

8. The last paragraph of Subsection 2.4.5.4.9 (As added in response to NRC Letter 095, RAI 02.04.05-10) will be replaced with the following paragraph:

   "As shown in Table 2.0-201 the DCD plant elevation for the LNP site is 100 ft. which is equivalent to 51 ft. NAVD88. The PMH maximum water surface elevation including wind-wave effect is 15.17 m (49.78 ft.) NAVD88. The highest PMH water surface elevation of 49.78 ft. NAVD88 provides 0.37 m (1.22 ft.) of margin to the DCD Maximum Flood Level site parameter of 51.0 ft. NAVD88."

9. In Table 1.9-201 "Regulatory Guide/FSAR Section Cross-References":

   Subsection numbers 2.4.5.4.4 and 2.4.5.4.9 will be added in the last column corresponding to the row for RG 1.59, with LMA LNP COL 1.9-1.

10. Table 2.0-201 "Comparison of AP1000 DCD Site Parameters and LNP Site Characteristics":

    On line item "Flood Level", third column, 4th Item "The maximum total ... 48.98 ft. NGVD 29." will be revised to read:

    "The maximum total (surge and wave action) water elevation from a PMH is 49.78 ft. NAVD88 or 50.78 ft. NGVD29."

    On line item "Flood Level", fourth column, 4th Item "FSAR Subsection 2.4.5.3.2" will be revised to read:

    "FSAR Subsection 2.4.5.4.9"

11. In the revised Subsection 2.4.16 (As revised in response to NRC Letter 095, RAI 02.04.05-10) the following new references will be added:

    2.4.5-226 National Oceanic and Atmospheric Administration (NOAA)'s web site for tides and currents online datum conversion chart , Link:http://tidesandcurrents.noaa.gov/data_menu.shtml?stn=8727520%20Cedar%20Key,%20FL&type=Datums, Used for Cedar Key, Florida, Accessed on June 26, 2011.

    2.4.5-227 National Oceanic and Atmospheric Administration (NOAA)'s web site for tides and currents Link: http://tidesandcurrents.noaa.gov/geo.shtml?location=8727520; for Cedar Key, Florida, station 8727520, Accessed on June 26, 2011.

    2.4.5-228 U.S. Army Corps of Engineers. Coastal Engineering Manual EM 1110-2-1100. Part VI Chapter 5. June 2006.

2.4.5-229 U.S. Army Corps of Engineers. Coastal Engineering Manual EM 1110-2-1100. Part II Chapter 2. June 2006.

Attachments/Enclosures:

Attachment 02.04.05-11-A: Revised Table 2.4.5-223

Attachment 02.04.05-11-B: New Figure 2.4.5-247

Attachment 02.04.05-11-A

Updated Table 2.4.5-223

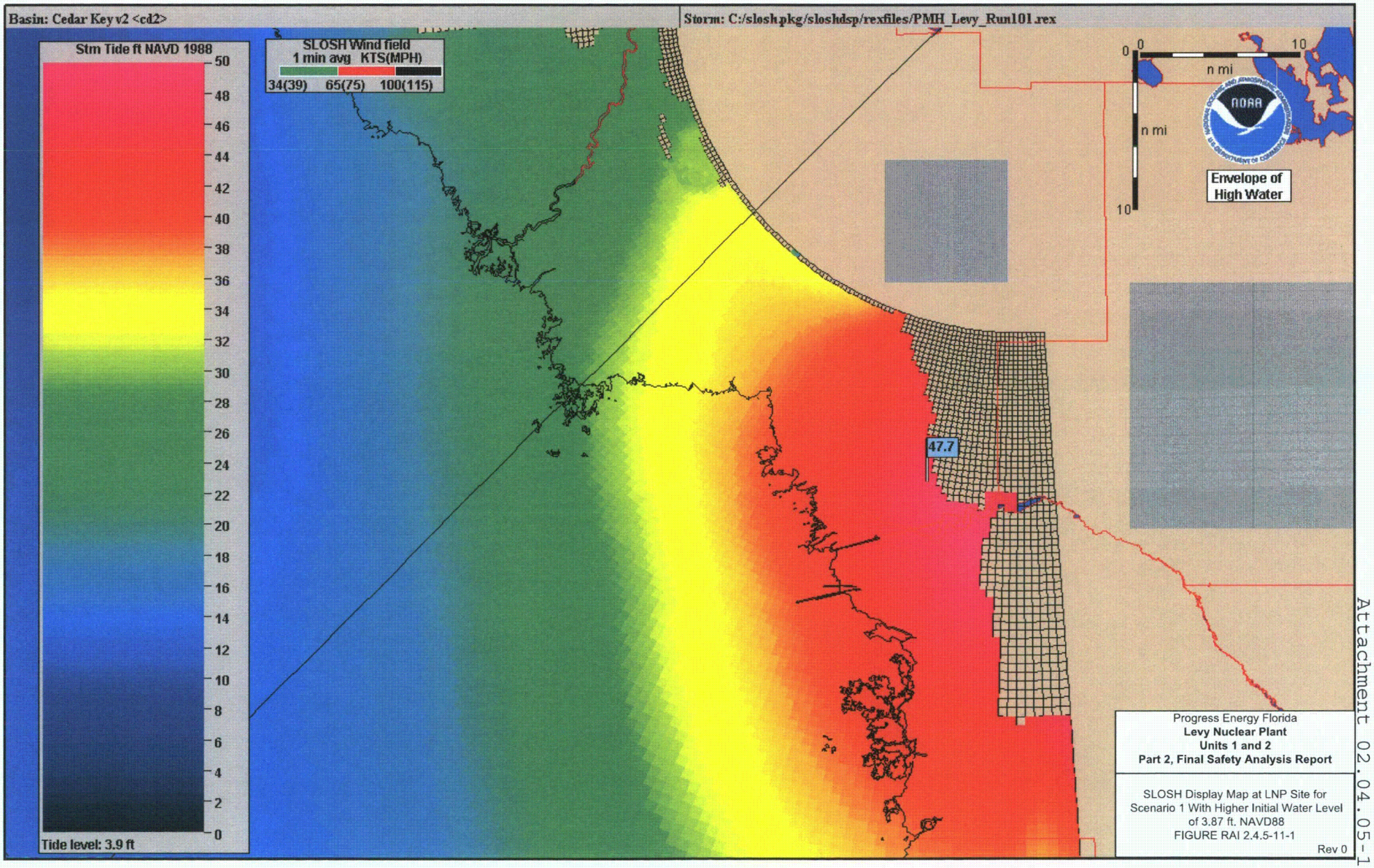
LNP COL 2.4-2

Table 2.4.5-223
Total PMH Surge elevation at LNP Site Including Wave Effects

Component	Units	Scenario 1	Scenario 2	Scenario 3	Scenario with Higher Initial Water Level of 3.87 ft. NAVD88
LNP Site Grade Elevation	FEET NAVD88	50.00	50.00	50.00	50.00
10 percent Exceedance Spring High Tide Elevation Including the Initial Rise.	FEET NAVD88	3.23	3.23	3.23	3.28
Long-term sea Level Rise	FEET	0.59	0.59	0.59	0.59
Initial Water Level	FEET NAVD88	3.82	3.82	3.82	3.87
SLOSH Surge Elevation With Initial Water Level Including 10 percent Exceedance High Tide, Initial Rise and Long-term Sea Level Rise	FEET NAVD88	47.70	47.30	46.80	47.70
Wave Setup	FEET	0.60	0.50	0.40	0.60
Wave Runup	FEET	1.48	0.90	0.23	1.48
TOTAL PMSS Including Wave Effects	FEET NAVD88	49.78	48.70	47.43	49.78

Attachment 02.04.05-11-B

New Figure 2.4.5-247



Attachment 02.04.05-11-B