NEXTera ENERGY
POINT BEACH

July 15, 2013 NRC 2013-0069

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
11555 Rockville Pike
Rockville, MD 20852

Point Beach Nuclear Plant, Units 1 and 2
Dockets 50-266 and 50-301
Renewed License Nos. DPR-24 and DPR-27

Supporting Documentation for July 22, 2013, Regulatory Conference to Discuss Inspection Report 05000266/2013011 and 05000301/2013011, Preliminary Yellow Finding

References: 1) U.S. Nuclear Regulatory Commission, Point Beach Nuclear Plant, Units 1 and 2 NRC Integrated Inspection Report 05000266/2013011 and 05000301/2013011; Preliminary Yellow Finding, dated June 18, 2013. (ML13169A212)

2) Point Beach letter NRC-2013-0054 Response to Inspection Report 050000266/2013011; Preliminary Yellow Finding, dated June 28, 2013. (ML13179A333)

On June 18, 2013, the Nuclear Regulatory Commission (NRC) provided NextEra Energy Point Beach, LLC (NextEra) with the results of the Temporary Instruction (TI) 2515-187, "Inspection of Near-Term Task Force Recommendation 2.3 Flooding Walk Downs," conducted at the Point Beach Nuclear Plant (PBNP) during the first quarter of 2013, describing a performance deficiency related to the PBNP implementation of certain procedures intended to mitigate postulated flooding events (Reference 1). The Reference 1 letter further informed NextEra that NRC had preliminarily determined that the significance of the identified performance deficiency was yellow.

On June 28, 2013, NextEra requested a Regulatory Conference to discuss the significance determination (Reference 2). The requested Regulatory Conference has been scheduled for July 22, 2013.

NextEra has thoroughly reviewed the issue raised in the Reference 1 letter, and has concluded that the Individual Plant Examination for External Events (IPEEE) contains estimates and assumptions that are overly conservative and it is not appropriate to use in the safety significance determination for this performance deficiency. Therefore, NextEra has performed substantial additional analyses utilizing more recent best-available information and modeling to more accurately determine the potential safety significance of the identified performance deficiency. Using the updated external flooding analysis and Probabilistic Risk Assessment

(PRA) models of the effects of postulated flooding is the correct tool for assessing the safety significance of the performance deficiency. . The results of the updated analyses clearly demonstrate that the safety significance of the performance deficiency is very low.

As requested in Reference 1, NextEra has provided both a summary description of the updated analyses and an explanation of the results in the Enclosure to this letter.

Attachment 1 contains the updated wave run-up analysis performed by NextEra's independent contractor. Attachment 2 is an updated safety significance determination analysis showing that the safety significance of the performance deficiency is very low, with margin. Attachment 3 provides an explanation of the PBNP design, showing that the running Service Water pumps would continue to operate following a loss of DC control power. Finally, Attachment 4 is an analysis of the rates of rise of Lake Michigan during various postulated flooding events, showing that PBNP would have more than eight weeks to respond to pre-storm level changes before the station's license basis flood level would occur.

NextEra looks forward to discussing these documents, together with our assessment of the very low safety significance of the performance deficiency, in greater detail during the upcoming Regulatory Conference.

This letter contains no new Regulatory Commitments and no revisions to existing Regulatory Commitments.

If you have any questions or require additional information, please contact Mr. Ron Seizert, Licensing Supervisor at (920)755-7500.

Very truly yours,

NextEra Energy Point Beach, LLC

Larry Meyer
Site Vice President

Enclosure

cc: Administrator, Region III, USNRC
Project Manager, Point Beach Nuclear Plant, USNRC
Resident Inspector, Point Beach Nuclear Plant, USNRC
Branch Chief, Plant Support, Division of Reactor Safety, Region III, USNRC

**ENCLOSURE**

**NEXTERA ENERGY POINT BEACH, LLC**
**POINT BEACH NUCLEAR PLANT**

**UPDATED FLOODING ANALYSIS AND SIGNIFICANCE DETERMINATION**

## Executive Summary

The NRC preliminary significance determination for the performance deficiency (Reference 1) was based on determining the change in Core Damage Frequency (CDF) using the Individual Plant Examination of External Events (IPEEE). NextEra has developed an updated, more accurate storm surge/wave run-up analysis which provides a more reliable analytical basis for assessing the associated safety significance of the postulated flooding event. The application of this more accurate approach shows that the identified performance deficiency is of very low safety significance, with margin.

## Description of the Updated Flooding Analysis and Resulting Significance Determination

The external flooding analysis for Point Beach Nuclear Plant (PBNP) contained in the station's IPEEE is dominated by overly conservative estimations and assumptions. These estimations and assumptions include the resultant water elevations, environmental conditions, equipment elevations, and site configuration and topography. The cumulative effect of these overly conservative assumptions results in significantly overestimating the impact of external flooding events, including wave run-up.

In order to better determine the safety significance of the identified performance deficiency, NextEra retained an expert independent engineering firm to update the station's external flooding analysis utilizing more recent and more accurate data. The updated analysis assumes the same initiating still water level frequencies as those used in the IPEEE. Further, the updated analytical model utilizes actual plant structures, configuration, topography, and near-shore bathymetric survey data in calculating the range of water levels and associated wave run-up conditions. The updated model included no off-setting credit for any of the flood/wave run-up mitigation actually in place at PBNP to determine water levels outside of the Turbine Building. The calculated water level provides the effective driving head for any potential water intrusion into the Turbine Building.

NextEra performed analysis of the potential water flow paths in the Turbine Building to determine the time it takes to accumulate sufficient water to impact safety significant equipment. A computer model was used to conduct this analysis, based on plant flow paths and rates, which were validated by walk-downs. Water flow rates were determined by calculating flow under doors, over curbs, and through other building characteristics. It would take greater than three hours for water outside the turbine building to impact the Residual Heat Removal (RHR) Pumps and their suction valves from Containment Sump B. However, for the purpose of simplification and to build additional margin into the analysis, the water was assumed to reach equilibrium inside and outside the buildings at time zero for the purpose of determining impact to equipment.

Finally, the calculated water level outside the Turbine Building was used as an input to the PBNP Probabilistic Risk Assessment (PRA) to evaluate the risk significance of the postulated flooding/wave run-up scenarios. This updated risk significance determination assumed that the external flooding wave run-up protection mitigation features described in the PBNP Final Safety Analysis Report (FSAR) were not in place.

The resulting significance determination, using the updated analysis and conservative assumptions described above, demonstrate that the safety significance of the performance deficiency is very low, with margin.

**Evaluation of Flood/Wave Run-Up, Water Levels and Potential Equipment Impacts**

The PBNP external flooding analysis had been updated to more accurately predict the potential flooding/wave run-up impact on PBNP and the result of the revised analysis was used to assess the potential impact of the calculated water levels on plant equipment.

The equipment impact analysis utilizes the same initial conditions contained in the FSAR, updated to include actual site topography, offshore bathymetry and the as-built shore-line configuration to calculate expected wave phenomena at various lake levels, including those of very long recurrence intervals (e.g., high lake levels). The resultant wave phenomena were then used as input data for the DELFT3D computer model, a state of the art model that can simulate both two dimensional (in either the horizontal or vertical plane) and three dimensional flow, to analyze Lake Michigan behaviors during external flooding events and to determine the resulting effect of wave run-up at PBNP.

The DELFT3D model has been accepted by industry experts and industry organizations including more than 70 countries world-wide that use DELTARES hydrology modeling programs and is being used extensively for post-Fukushima flood hazard analyses. The DELFT3D model has also been recently used by several other nuclear plants to demonstrate compliance with NRC requirements, including:

- South Texas Project, Units 3 and 4 COLA and FSAR (Delft3D-FLOW) for breach and wave modeling
- Turkey Point Units 6 and 7 (Delft3D-FLOW), tsunami wave analysis
- Turkey Point Units 3 and 4 flood hazard reevaluation
- Nine Mile Point (NMP) (DELFT-SWAN), for near shore wave heights and periods
- Calvert Cliffs used DELFT for storm surge for COLA and flood hazard reevaluation
- Victoria County Station Early Site Permit Application (Delft3D-FLOW) for Cooling Basin Breach Analysis

Accordingly, NextEra is confident that its updated engineering evaluation of potential wave run-up effects at PBNP provides the best available information to use for safety significance determinations.

**Comparison of the Updated Storm Surge/Wave Run-up Analysis to the PBNP IPEEE Analysis**

The most limiting flooding from Lake Michigan is a function of the still water lake level plus wind generated waves. To estimate the frequency of flooding at PBNP, the IPEEE utilized a statistical frequency distribution that was estimated from Lake Michigan gauge data. This still water lake level has also been used in the updated Storm Surge/Wave Run-up Analysis (Updated Wave Run-Up Analysis) as a starting point for the modeling.

However, although the IPEEE utilizes an estimated wave run-up, it does not describe the estimation methodology utilized to determine its results. It is clear, however, that the IPEEE estimation methodology does not take into account site specific shoreline and near-shoreline configurations, which are very important for an accurate determination of the storm effects. The Updated Wave Run-Up Analysis, on the other hand, utilizes state of the art modeling and analytical methods, and updated site-specific input data to determine wave set-up and wave run-up.

Like the IPEEE, the Updated Wave Run-Up Analysis also utilizes conservative assumptions for evaluating the depth of water between the Turbine Building and Pumphouse. Specifically, both analyses assume the following:

- Yard drains do not contribute to runoff from flooding.
- There are no relief paths from flooding (*e.g.*, Turbine Building floor drains and Pumphouse relief paths).

The IPEEE utilizes simplified, conservative estimates for the amount of water that would accumulate between the Turbine Building and the Pumphouse from wave run-up, but again the IPEEE does not describe the estimation methodology used to determine the presented results. The Updated Wave Run-Up Analysis incorporates license basis deep water wave heights, more extensive local and offshore bathymetry, and the as-built shore-line configuration and characteristics to calculate expected wave response at various still water levels.

In determining the equipment impacted, the internal water levels were assumed conservatively to equal the calculated outside water levels at time zero.

A comparison of the overly conservative IPEEE and the more accurate Updated Wave Run-Up Analysis showing the differences in calculated water level between the PBNP Turbine Building and Pumphouse, is presented below.

**Summary of Water Levels IPEEE vs. PBNP Updated Analysis**

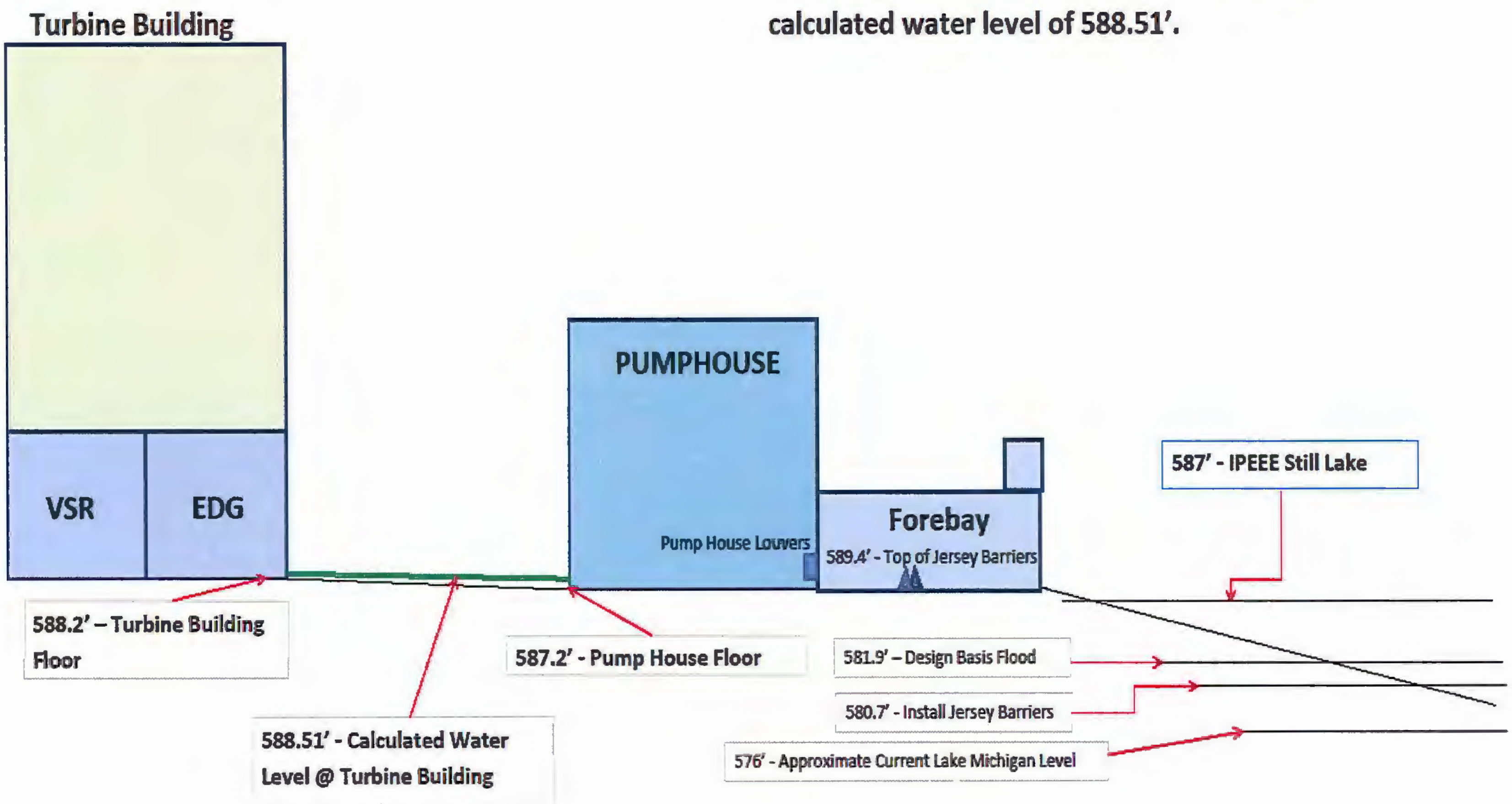
| Still Water Level (ft-IGLD 1955) | Frequency of Still Water Level (yr-1) | Water Level Extrapolated from the IPEEE Estimates (ft-IGLD 1955) | Calculated Water Level (With Wave Run-up) (ft-IGLD 1955) | Resulting Water Level in Turbine Building (inches above floor) |
|---|---|---|---|---|
| 583.00 | 1.4E-02 | 587.60 | 583.96 | 0 |
| 585.00 | 3.2E-04 | 588.37 | 585.84 | 0 |
| 586.00 | 5.4E-05 | 588.73 | 587.24 | 0 |
| 587.00 | 4.2E-06 | 589.24 | 588.51 | 3.7" |
| 587.64 | 9.9E-07 | 589.53 | 588.89 | 8.3" |

These results demonstrate that the IPEEE was overly conservative in estimating the water level between the Turbine Building and Pumphouse and is not appropriate for use in assessing safety significance because it is not the best-available information. The result of the Updated Wave Run-Up Analysis is appropriate input to determine the safety significance of the performance deficiency.

The following Figure shows the levels resulting from the updated analysis.

Simplified PBNP Plant Elevation with Calculated Water Level (Not to Scale)

IPEEE Describes Lake Level of 587' and Wave Run-Up to 596' with Frequency of 4.2E-6. Updated analysis of lake level of 587' with wave run-up, results in calculated water level of 588.51'.



## Results of the Updated Wave Run-Up Analysis

The Updated Wave Run-Up Analysis shows that the calculated water levels between the Turbine Building and Pumphouse would be 588.51 feet of water IGLD 1955 for the event frequency of 4.2E-06/year—much less severe than estimated in the IPEEE.

The detailed engineering evaluation supporting NextEra's Updated Wave Run-Up Analysis is provided in Attachment 1.

## Point Beach Revised External Flood PRA

The still water level flood exceedance frequencies from the IPEEE were utilized in NextEra's updated analysis so that the impacts from this updated analysis could be compared to the same flood frequencies used by the NRC. With the results from the updated external flooding analyses, conditional core damage probability was calculated using the updated RG 1.200 internal events model.

Point Beach letter NRC-2013-0054 Response to Inspection Report 50000266/2013011 (Reference 2) provided a list of potentially vulnerable equipment impacted by accumulating water up to 589.2 feet (IGLD 1955). NextEra's updated analysis conservatively assumed an internal water level of 588.51 feet (IGLD 1955) based on the same calculated water level outside the Turbine Building. This internal water level impacts only the Residual Heat Removal (RHR) pumps and RHR pump suction from Containment Sump B, therefore this equipment is not available in the revised PRA evaluation.

Based on the equipment impacts described above, the PRA results indicate that the risk due to external flooding (without barriers) is very low, with a Core Damage Frequency (CDF) equal to 2.83E-07/year. The associated change in risk with and without barriers was also determined to be very low, with a change in CDF equal to 7E-09/year.

The NRC's preliminary significance determination of the performance deficiency was based on the 1995 Point Beach IPEEE inputs and assumptions. This determination calculated a conditional core damage probability of 2.7E-02/yr. and a change in CDF of 1.8E-05/yr. NextEra's updated analysis demonstrates that the IPEEE assumptions of wave run-up levels and final water levels at and in the Turbine Building were overly conservative. Principle contributors to the differences in overall safety significance include the site topography, the existing site layout and features, equipment elevations, and environmental conditions. Each of these items has been considered in the updated detailed modeling described herein.

Attachment 2 provides the supporting information associated with the PRA evaluation.

## Continued Service Water Pump Availability with Loss of DC Control Power

A significant contributor to the difference between the NextEra and the NRC staff estimated change in CDF with failure of equipment up to 589.2 feet IGLD 1955 is the PRA modeling of Service Water (SW) pump availability upon loss of DC control power. NRC's SPAR model contains a presumption that the SW pump becomes unavailable upon loss of DC power.

The SW pumps remain available throughout this event. If operating at the time DC power is lost, the SW pumps would continue to operate, because DC power is required to open the breaker supplying AC power to the pumps. In other words, if DC power is lost, the breakers cannot open to turn off the running SW pumps. If the pumps are not operating at the time DC power is lost, operators can start pumps by either realigning DC control power supplies and/or through local manual operation of their respective breakers. Both actions are directed by existing site procedures upon loss of DC power.

The actions for switching to the alternate DC control power source and local operation of SW pumps, including flow control, are directed by site procedures. These actions were re-validated by walk-downs with Auxiliary Operators and observed by independent Nuclear Oversight (NOS) observations. The validation indicated that the actions can be completed within one hour. Finally, these actions are included in Operator training and evaluation programs.

Attachment 3 provides the supporting information associated with SW pump availability.

## Lake Michigan Level Rate of Rise

Lake Michigan is a very large body of water and changes in level are very slow in comparison to river level changes. The PBNP original Final Facility Description Safety Analysis Report (FFDSAR) section discussing flooding does not address a rate of rise for the lake, so an evaluation was performed to establish the time that is available to respond to rising lake levels.

The rate of rise in lake level was calculated using National Oceanographic and Atmospheric Administration (NOAA) historical lake data for the 95 year period from 1918 through 2013. During this period, the greatest increase in Lake Michigan level during a single month was 0.85 ft. The evaluation concluded that the time available to respond to rising Lake Michigan pre-storm levels would be approximately eight weeks from the level at which PBNP procedures require wave run-up barrier construction initiation (580.7 feet IGLD 1955) until conditions for the license basis maximum wave run-up could be reached. With respect to the identified performance deficiency, this eight week time period provides significant opportunity to identify and correct deficiencies with flood barriers. The updated analysis performed in support of this evaluation contains significant margin, including that the PRA analysis does not account for the significant time available for Operators to take actions in response to rising lake levels.

Attachment 4 provides the supporting information associated with the rate of change in Lake Michigan water level and required Operator actions.

## Conclusion

The updated external flooding analysis shows that the frequency of reaching a calculated water level that impacts safety related equipment is very low. The PRA analysis, using the calculated water levels, determined that affected equipment results in a very low CDF. Therefore, the failure to establish adequate procedure requirements to implement external flooding wave run-up protection features as described in the FSAR has very low safety significance, based on the calculated change in CDF of 7E-09/yr.

The Individual Plant Examination for External Events (IPEEE) contains estimates and assumptions that are overly conservative and it is not appropriate to use in the safety significance determination for this performance deficiency. The IPEEE estimated that water levels would be substantially higher with the same input assumptions, because of the many simplifications that were used at the time. With more accurate site information and a more rigorous tool for analysis, the conservatisms of the IPEEE were shown to be excessive.

**Attachments**

1) Calculated Average External Water Levels at Turbine Building
2) Point Beach Revised External Flood Safety Significance Determination
3) Continued Service Water Pump Availability with Loss of DC Control Power
4) Lake Michigan Level Rate of Rise

# ATTACHMENT 1

**NEXTERA ENERGY POINT BEACH, LLC**
**POINT BEACH NUCLEAR PLANT**

**CALCULATED AVERAGE EXTERNAL WATER LEVELS AT TURBINE BUILDING**

**20 Pages Follow**

| ENERCON | CALCULATION COVER SHEET | CALC. NO. FPL-076-CALC-004 |
|---|---|---|
| | | REV. 1 |
| | |  |

| Title: | Calculated external average water levels at turbine building | Client: NextEra Energy (NEE) |
|---|---|---|
| | | Project: FPLPB025 |

| Item | Cover Sheet Items | Yes | No |
|---|---|---|---|
| 1 | Does this calculation contain any assumptions that require confirmation?<br>(If **YES**, identify the assumptions)____________ | | X |
| 2 | Does this calculation serve as an "Alternate Calculation"? (If **YES**, identify the design verified calculation.)<br>**Design Verified Calculation No.**____________ | | X |
| 3 | Does this calculation supersede an existing calculation? (If **YES**, identify the superseded calculation.)<br>**Superseded Calculation No.**____________ | | X |

**Scope of Revision:**

Text revised on Page 15

**Revision Impact on Results:**

No impact to results. Text only.

| | | | |
|---|---|---|---|
| **Study Calculation** | ☐ | **Final Calculation** | ☒ |
| **Safety-Related** | ☒ | **Non-Safety Related** | ☐ |

*(Print Name and Sign)*

| | | | |
|---|---|---|---|
| **Originator:** | **Shaun W. Kline** | Shaun W Kline | **Date:** 07/15/2013 |
| **Design Verifier:** | **Justin Pistininzi** | | **Date:** 07/15/2013 |
| **Approver:** | **Paul Martinchich** | | **Date:** 07/15/2013 |

| ENERCON | CALCULATION<br>REVISION STATUS SHEET | CALC. NO.<br>FPL-076-CALC-004<br>REV. 1<br>PAGE NO. 2 of 20 |
|---|---|---|

**CALCULATION REVISION STATUS**

| REVISION | DATE | DESCRIPTION |
|---|---|---|
| 1 | July 15, 2013 | Initial Issue |

**PAGE REVISION STATUS**

| PAGE NO. | REVISION | PAGE NO. | REVISION |
|---|---|---|---|
| All pages | 1 | | |

**Appendix/Attachment REVISION STATUS**

| Attachment NO. | PAGE NO. | REVISION NO. | Attachment NO. | PAGE NO. | REVISION NO. |
|---|---|---|---|---|---|
| All Attachments | All Pages | 1 | | | |

<table>
<tr><td rowspan="3">ENERCON</td><td rowspan="3">CALCULATION<br>DESIGN VERIFICATION PLAN<br>AND SUMMARY SHEET</td><td>CALC. NO.<br>FPL-076-CALC-004</td></tr>
<tr><td>REV. 1</td></tr>
<tr><td>PAGE NO. 3 of 20</td></tr>
</table>

**Calculation Design Verification Plan:**

Apply CSP Number 3.01, Revision 6, Section 4.5.a, Design Review Method and to include at a minimum:

1. Review and verify the design inputs, references and tables to ensure that the Calculations Results, as they conform to the design methodology and design guidance, are correct.

***(Print Name and Sign for Approval – mark "N/A" if not required)***

| | |
|---|---|
| **Approver:** **Paul Martinchich** | **Date:** 07/15/2013 |

**Calculation Design Verification Summary:**

After reviewing this calculation and all related documents for Revision 0, I have come to the following conclusions:

1. The methodology, design inputs and approach are appropriate for the derivation of all calculated results.
2. The results of the Calculation are reasonable based on verified input values.
3. The report text and general flow of the document is clear and concise.

**Based on the above summary, the calculation is determined to be acceptable.**

***(Print Name and Sign)***

| | |
|---|---|
| **Design Verifier: Justin Pistininzi** | **Date:** 07/15/2013 |
| **Others: N/A** | **Date:** |

| ENERCON | CALCULATION DESIGN VERIFICATION CHECKLIST | CALC. NO. FPL-076-CALC-004 |
|---|---|---|
| | | REV. 1 |
| | | PAGE NO. 4 of 20 |

| Item | Cover Sheet Items | Yes | No | N/A |
|---|---|---|---|---|
| 1 | Design Inputs - Were the design inputs correctly selected, referenced (latest revision), consistent with the design basis and incorporated in the calculation? | X | | |
| 2 | Assumptions – Were the assumptions reasonable and adequately described, justified and/or verified, and documented? | X | | |
| 3 | Quality Assurance – Were the appropriate QA classification and requirements assigned to the calculation? | X | | |
| 4 | Codes, Standard and Regulatory Requirements – Were the applicable codes, standards and regulatory requirements, including issue and addenda, properly identified and their requirements satisfied? | X | | |
| 5 | Construction and Operating Experience – Has applicable construction and operating experience been considered? | | | X |
| 6 | Interfaces – Have the design interface requirements been satisfied, including interactions with other calculations? | X | | |
| 7 | Methods – Was the calculation methodology appropriate and properly applied to satisfy the calculation objective? | X | | |
| 8 | Design Outputs – Was the conclusion of the calculation clearly stated, did it correspond directly with the objectives and are the results reasonable compared to the inputs? | X | | |
| 9 | Radiation Exposure – Has the calculation properly considered radiation exposure to the public and plant personnel? | | | X |
| 10 | Acceptance Criteria – Are the acceptance criteria incorporated in the calculation sufficient to allow verification that the design requirements have been satisfactorily accomplished? | X | | |
| 11 | Computer Software – Is a computer program or software used, and if so, are the requirements of CSP 3.02 met? | X | | |

COMMENTS:

*(Print Name and Sign)*

| | |
|---|---|
| **Design Verifier: Justin Pistininzi** [signature] | **Date:** 07/15/2013 |
| **Others: N/A** | **Date:** |

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# 1. Purpose and Scope

This calculation is performed under NextEra Energy (NEE) contract order 02306247 to determine a(n)

- average water level proxy at the turbine building for the PRA leakage analysis, and
- maximum water level proxy at the turbine building for a structural loading analysis on the turbine building's rollup doors and the jersey barriers

due to effects of storm surge in Lake Michigan at the Point Beach Nuclear Plant (PBNP).

The purpose of this calculation is to provide a calculation by empirical relationship of incident wave runup elevations beyond the standing water elevation, which is comprised of the still water level (SWL) and wave setup (FEMA, 2005; Dean and Dalrymple, 2007; USACE, 2011). The initial SWL is prescribed and the wave setup is computed by the DELFT3D model (ENERCON, 2013a). Wave runup includes many simultaneous processes, and is the sum of static wave setup, dynamic wave setup, and incident wave runup (swash), as shown in Figure 1-1. To adequately address the water elevations on critical infrastructure at PBNP, an empirical relationship computation of the latter parameter (incident wave runup) is required. Proxies for critical total water elevations (which include the combined effects of SWL, wave setup, and individual wave swash) on the turbine building are also computed.

Results and conclusions from this calculation can be used to determine the effect of waves and flooding on the turbine building at PBNP. For instance, the calculated average water levels can be used to compute water leakage into the building and the maximum water level can be used to determine the highest forces against the turbine building rollup doors.

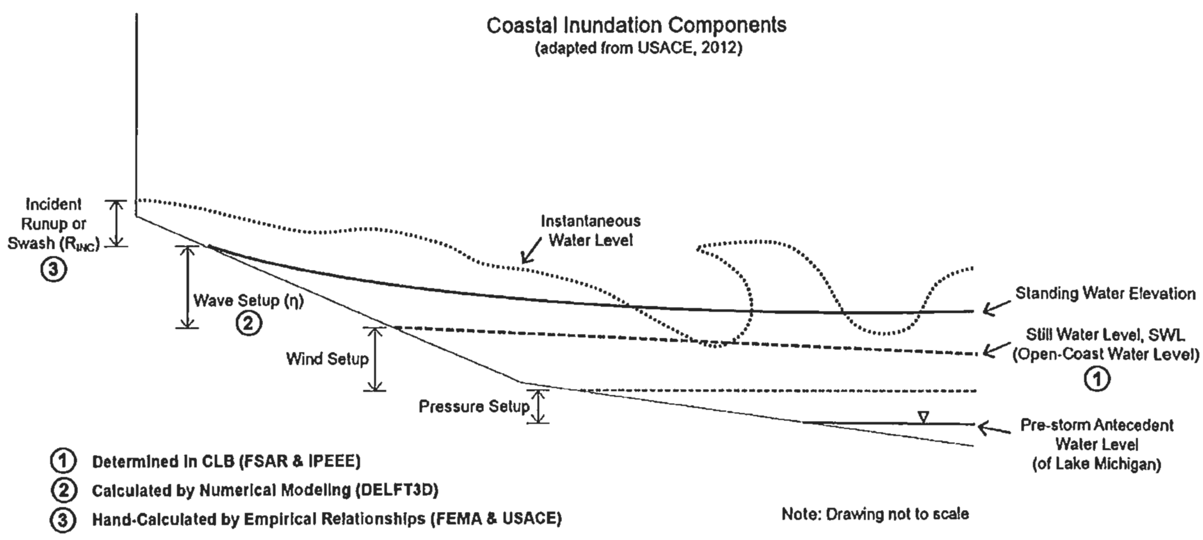



**Figure 1-1: Coastal Inundation Components**

# 2. Summary of Results and Conclusions

Conservative parameters were used whenever available in this analysis. Where conservative values were not available, reasonable inputs in accordance with industry standard practice and justification were used. Therefore, the results of this analysis are bounding for PBNP.

This analysis first considered incident wave runup, which empirically has been shown to be a function of deep-water characteristics and beachface slope (Stockdon, 2006; FEMA, 2005; Dean and Dalrymple, 2007; USACE, 2011), to help determine a total water elevation above the still water level and wave setup components modeled in DELFT3D (ENERCON, 2013a). Beach/beachface characteristics and breaking wave locations were found near the southwest corner of the pump house adjacent to the turbine building (observation point BP2, see Figure 2-1), where ponding occurs in the most severe cases (ENERCON, 2013a).

We also provide a water elevation proxy considering the mean setup, rather than the peak surge level simulated in DELFT3D (ENERCON, 2013a). The calculated average setup method produces a maximum mean water level proxy of 0.31 feet on the turbine building (588.51 ft-IGLD) during the worst-case IPEEE still water level elevation of 587 ft-IGLD55 (PBNP, 1995; PBNP, 2013). This value can be used to determine the effective water leakage experienced into the turbine building from pooling water against the doors.

We find an incident wave runup greater than 1.65 feet will occur for less than 2% of the storm duration (ENERCON, 2013a). Accordingly, a wave bore exceeding 2.19 feet (590.39 ft-IGLD55) could be expected less than 2% of the storm at the turbine building in the most severe IPEEE still water level case of 587 feet-International Great Lakes Datum 1955 (henceforth IGLD55) (PBNP, 1995; PBNP, 2013). For an initial water level of 586 ft-IGLD, maximum runup may reach the turbine building, but mean water level proxies indicate no persistent water present. This calculation can support analyses of door, building, and barrier stability in the presence of increased water level and/or wave attack.

The results in this calculation are applicable for a small, yet critical range of the turbine building near the southwest corner of the pump house. Conservative methods were applied to account for uncertainties related to the inputs. This incident runup calculation and total water elevation proxies can be applied to other PBNP locations, but the methodology and assumptions presented should be considered first. The total water elevations are expected to be lower at other PBNP turbine building locations of interest, since the maximum setup of the observations in Figure 2-1 was selected for this analysis (ENERCON, 2013a).

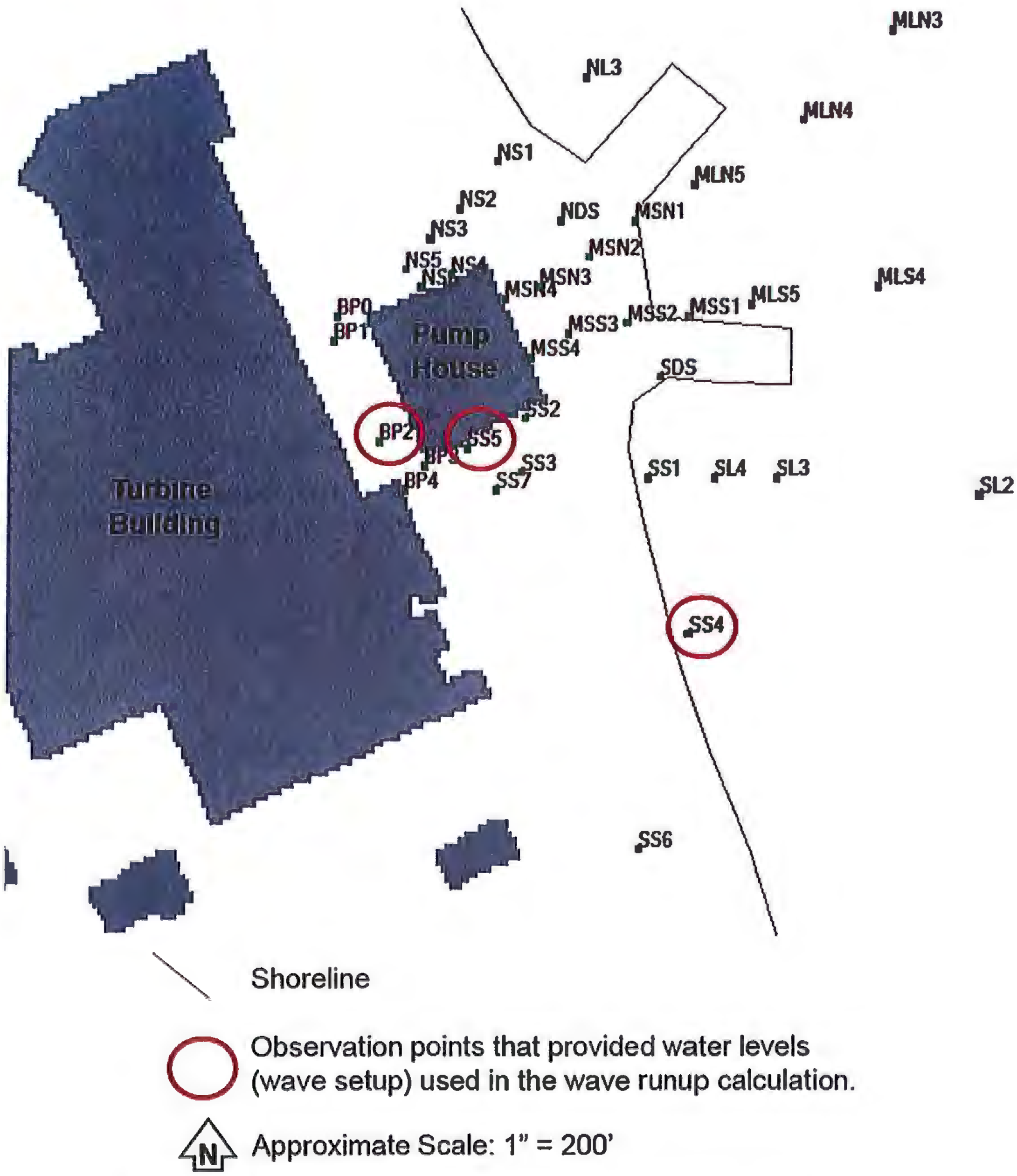



**Figure 2-1: Observation Points at Point Beach Nuclear Plant (PBNP).**

## 3. References

3.1 **Dean and Dalrymple, 2007**, "Water Wave Mechanics for Engineers and Scientists," World Scientifc, 353 pp.

3.2 **Deltares, 2012**, Deltares Systems, "DELFT3D-WAVE User Manual," updated 2012.

3.3 **ENERCON, 2013a**, ENERCON Services, Inc. (ENERCON), "DELFT3D Modeling of Surge and Wave Runup, Revision 0," Calculation Number FPL076-CALC-003, 2013.

3.4 **ENERCON, 2013b**, ENERCON Services, Inc. (ENERCON), "Mean Wave Flow, Revision 0," Calculation Number FPL-076-CALC-006, 2013.

3.5 **FEMA, 2005**, Federal Emergency Management Administration (FEMA), "Guidelines and Specifications for Flood Hazard Mapping Partners," D.4.5 Wave Setup, Runup, and Overtopping.

3.6 **FEMA, 2007**, Federal Emergency Management Administration (FEMA), "Guidelines and Specifications for Flood Hazard Mapping Partners," D.2.8 Wave Runup, and Overtopping.

3.7 **FEMA, 2012**, Federal Emergency Management Administration (FEMA), " FEMA Great Lakes Coastal Guidelines, Appendix D.3 Update," D.3 Coastal Flooding Analyses and Mapping: Great Lakes.

3.8 **Mase, 1988**, "Spectral Characterisitcs of Random Wave Runup," *Coastal Engineering*, Vol. 12, No. 2, pp.175-189.

3.9 **Melby, 2012**, United States Army Corps of Engineers, Coastal and Hydraulic Laboratory, "Wave Runup Prediction for Flood Hazard Assessment (Draft)."

3.10 **PBNP, 1995**, Point Beach Nuclear Plant (PBNP), "Point Beach Nuclear Plant Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities – Summary Report," Wisconsin Electric Power Company, June 30, 1995.

3.11 **PBNP, 2013**, Point Beach Nuclear Plant (PBNP) Design Information Transmittal (DIT), Modification Process, "Still Water Elevations (IGLD55) to be used for the FPL-076 series of calculations in the PBNP wave analysis to match up with existing event probabilities," July 11, 2013.

3.12 **S&L, 1967**, Sargent and Lundy (S&L), "Maximum Deep Water Waves & Beach Run-up at Point Beach," 1967.

3.13 **Stockdon, 2006**, "Empirical parameterization of setup, swash, and runup," Coastal Engineering, Vol. 53, No. 2, pp. 573-588.

3.14 **USACE, 2011**, United States Army Corps of Engineers (USACE), "Coastal Engineering Manual," EM 1110-2-1100, updated 2011.

3.15 **USACE, 2012**, United States Army Corps of Engineers (USACE), "Statistical Analysis and Storm Sampling for Lakes Michigan and St. Clair," 2012.

# 4. Assumptions

4.1 No reduction factors due to differing surface roughnesses have been applied. The beach/beachface extent includes predominately sand, grass, gravel, and asphalt, which require no reduction (FEMA, 2005). Applying no reduction factors is the most conservative approach for wave runup calculations.

4.2 The conversion from 2% incident wave runup elevation to 50% incident wave runup elevation is based on an empirical relationship applied to total runup for irregular waves (Mase, 1988; USACE, 2011). It is assumed that the conversion applies equivalently for wave setup and incident runup and that the conversion is valid for waves modeled in FPL-076-CALC-003. Given the conservative method of calculating incident runup and the lack of accepted industry standards for calculating mean incident runup heights, this is a reasonable approach.

4.3 An effective runup elevation, or average elevation of the wave runup bore over its entire wavelength (wave period), is determined by assuming the runup bore maintains a sinusoidal shape above the mean water surface, which is a conservative approach for a linear wave form. An effective (average) height for this bore is used to provide a proxy for mean effective incident runup elevations. Although breaking waves become nonlinear and lose their sinusoidal shape (Dean and Dalrymple, 2007), this approach provides an approximation of the average (effective) runup height. It is a reasonable and conservative approach given the lack of industry standard in the derivation of such a parameter.

4.4 Wave setup is assumed to be a constant value near the turbine building. The maximum or mean water level at observation points (BP2, SS4, or SS5) near the turbine building are used to determine the wave setup value (height above the initial still water level). This calculation provides a value of water levels near the southwest corner of the pump house and may differ at other PBNP locations. However, the calculated level bounds the expected level at the turbine building.

# 5. Design Inputs

The design inputs are listed in Table 5-1 below. A maximum wave period of 10 seconds was used to account for the maximum wind-generated periods determined in FPL-076-CALC-003 (Enercon, 2013a).

**Table 5-1: Incident Wave Runup and Water Elevation Calculation Inputs**

| | Value | Units | Source(s) |
| --- | --- | --- | --- |
| $H_0$ | 23.5 | ft | ENERCON, 2013a |
| T | 10 | s | ENERCON, 2013a |
| Wave Direction | 120 | degrees | ENERCON, 2013a |
| Wind Direction | 90 | degrees | ENERCON, 2013a |
| m | 0.025 | ft/ft | PBNP, 2012<br>S&L, 1976<br>ENERCON, 2013a |
| SWL | 587.64<br>587<br>586<br>585<br>583 | ft- IGLD55 | PBNP, 1995<br>PBNP, 2013 |

# 6. Methodology

## 6.1 Runup

Runup (R) is the maximum elevation of wave uprush above the still water level, consisting of wave setup ($\eta$), the elevation of the water surface due to wave action, and oscillatory wave swash from breaking waves (USACE, 2011):

$$R = \eta + R_{INC} \qquad \text{(FEMA, 2005, D.4.5-1)}$$

where:

R = total runup (ft)
$\eta$ = combined static and dynamic wave setup (ft)
$R_{INC}$ = incident wave runup due to oscillatory wave swash (ft)

Physically, R is the local maximum in water elevation (USACE, 2011). No reliable theoretical formulations for runup exist currently because the controlling processes are complex and nonlinear. Rather, empirically derived relationships are used to estimate runup (USACE, 2011).

Many of these formulae relate total runup (R) to wave characteristics and beachface morphology (i.e. slope). For the purposes of this analysis, however, such a computation would not be required. Computed DELFT3D water levels (in FPL-076-CALC-003) include wave setup ($\eta$). Thus, the only additional increase to the maximum water elevation is the oscillatory wave swash, $R_{INC}$.

## 6.2 Incident wave runup and maximum water levels for stability analysis

FEMA defines the 2% incident wave runup ($R_{INC\,2\%}$), the oscillatory water elevations attributable to wave swash that is to exceed less than 2% of the time, as

$$R_{INC\,2\%} = 0.6\,\frac{m}{\sqrt{\frac{H_0}{L_0}}}H_0 \qquad \text{(FEMA, 2005, D.4.5-11)}$$

where:

$R_{INC\,2\%}$ = incident wave runup elevation beyond water surface exceeded by <2% of waves (ft)
m = beach slope (dimensionless)
$H_0$ = deep-water wave height (ft)
$L_0$ = deep-water wavelength (ft)

Thus, maximum incident runup occurs on steep beaches for large, long waves. Equation D.4.5-11 was developed empirically along the Pacific coast (FEMA, 2005). It was chosen for this calculation for two major reasons: 1) it provided a clear evaluation of the incident wave runup, rather than the combined effects of incident runup and setup, and 2) it is conservative approach, since frictional dissipation over a wide, gently-sloped dissipative beach, like Lake Michigan, would yield lower incident runup heights than the steeper, narrower surf zones of the Pacific coast (Stockdon, 2006).

FEMA (2012) does provide unofficial (draft) guidelines for the flood mapping on the Great Lakes, and promotes the use of Melby's (2012) compilation of wave runup formulations when applicable. However, equations therein reference a total runup elevation (Melby, 2012, Figure 1). FEMA (2012) defines runup as a "statistic associated with a group of waves or a particular storm," indicating that the equations do not separate setup from individual wave swash. Runup formulations for structures are also provided (FEMA, 2012), but are irrelevant when the largest waves modeled in FPL-076-CALC-003 break far seaward of PBNP and do not directly impact any vertical infrastructure. Hence, Equation D.4.5.11 is the best known, applicable, yet conservative estimate of only incident wave swash.

The beach is defined as the area between wave breaking and the landward extent of wave runup (FEMA, 2005). The landward extent of the beach is defined as the turbine building near the southwest corner of the pump house (observation point BP2) where ponding was predicted in the most severe cases (ENERCON, 2013a; Table 7-2). Additionally, the turbine building would interrupt runup flow and is the structure of top importance. The elevation of the bottom of the turbine building door is 588.2 ft-IGLD55.

At PBNP, the seaward extent of the beach was located where wave breaking occurred, as predicted by the DELFT3D-WAVE model (ENERCON, 2013a). Larger waves began breaking ~700 feet from the turbine building, whereas smaller wind-generated waves, which produce significantly less incident runup broke approximately ~250 feet from the BP2 observation point (ENERCON, 2013a).

In deep-water, wavelengths are directly related to the wave period (T):

$$L_0 = \frac{gT^2}{2\pi} \qquad \text{(FEMA, 2005, D.4.5-10)}$$

where:

T = wave period (s)

Incident runup heights thus become a function of deep-water wave height and period, as well as the beach slope.

The largest incident wave runup values occur on steep beachfaces for waves that are large ($H_0$) and long (T). Thus, the simulations with the largest $H_0$ and T values along the deep-water boundaries would produce the highest incident wave runup elevations. The water elevation given by SWL+$\eta_{MAX}$+$R_{INC\ 2\%}$ ($WL_{2\%}$) is a proxy for the maximum water level reached during the storm. It is the value used in stability analysis used of the turbine building's rollup doors and jersey barriers.

6.3 Effective incident runup to determine external water levels on turbine building

The incident wave runup ($R_{INC}$) is only an instantaneous water elevation (Figure 6-1). To better understand the time-averaged, mean total water level, an average runup elevation proxy may be developed. At PBNP, the mean water level is an important factor in the leakage experienced in the turbine building. After a wave breaks, it continues as a bore up the beachface until gravity limits its upward swash rush or it is interrupted by a hard structure. For sinusoidal bores that are equally spaced (i.e. the length of the bore is equal to the space between bores, the mean elevation can be given in two parts. The first part is the same calculation performed in FPL-076-CALC-006, and the result from that analysis (Equation 6-2 in FPL-076-CALC-006) is provided here:

$$A_{BORE} = \frac{R_{INC\,2\%}L}{2\pi} \qquad (6\text{-}1)$$

where:

$A_{BORE}$ = area under the wave bore (ft$^2$)
L = wavelength (ft)

The average height of that bore is given by dividing by half of the nearshore wavelength, L/2 (see Figure 6-1):

$$\overline{h_{BORE}} = \frac{A_{BORE}}{L/2} = \frac{R_{INC\ 2\%}}{\pi} \qquad (6\text{-}2)$$

where:

$\overline{h_{BORE}}$ = average height of the bore (ft)

Note that the height of the wave bore is not a true wave height; rather, it is a wave amplitude, since the trough of the bore is neglected. The second part of the calculation is the space between bores, which is assumed to be at the standing water elevation (Figure 6-1). Since the 'bore' has a height of zero in this range, the overall effective height of the bore, and subsequently of the incident runup ($R_{EFF\ 2\%}$), if the next bore is assumed to follow the profile in Figure 6-1, is given by:

$$R_{EFF\ 2\%} = \frac{\bar{h}_{BORE}}{2} = \frac{R_{INC\ 2\%}}{2\pi} \qquad (6\text{-}3)$$

where:

$R_{EFF\ 2\%}$ = effective runup height (ft)

This value ($R_{EFF\ 2\%}$) is used as a proxy of the wave runup provided by the incident runup ($R_{INC}$) predicted by Equation D.4.5-11 from FEMA (FEMA, 2005). This approximation is a conservative approach of the linear wave profile since the wave trough is neglected.

Additionally, the USACE (2011) provides empirical formulas for various total runup thresholds (e.g. $R_{MAX}$, $R_{2\%}$, $\bar{R}$) for irregular waves (USACE, 2011). If incident wave runup and wave setup, the two components of total wave runup, are assumed to vary equally (i.e. increase/decrease by the same percentage) between these various thresholds, then a comparison of the empirical formulas will allow for a conversion of $R_{EFF\ 2\%}$, derived from applying Equation 6-3 to an $R_{INC\ 2\%}$ computation, to an equivalent $R_{EFF\ 50\%}$ proxy for $\bar{R}$, the mean runup calculation (USACE, 2011):

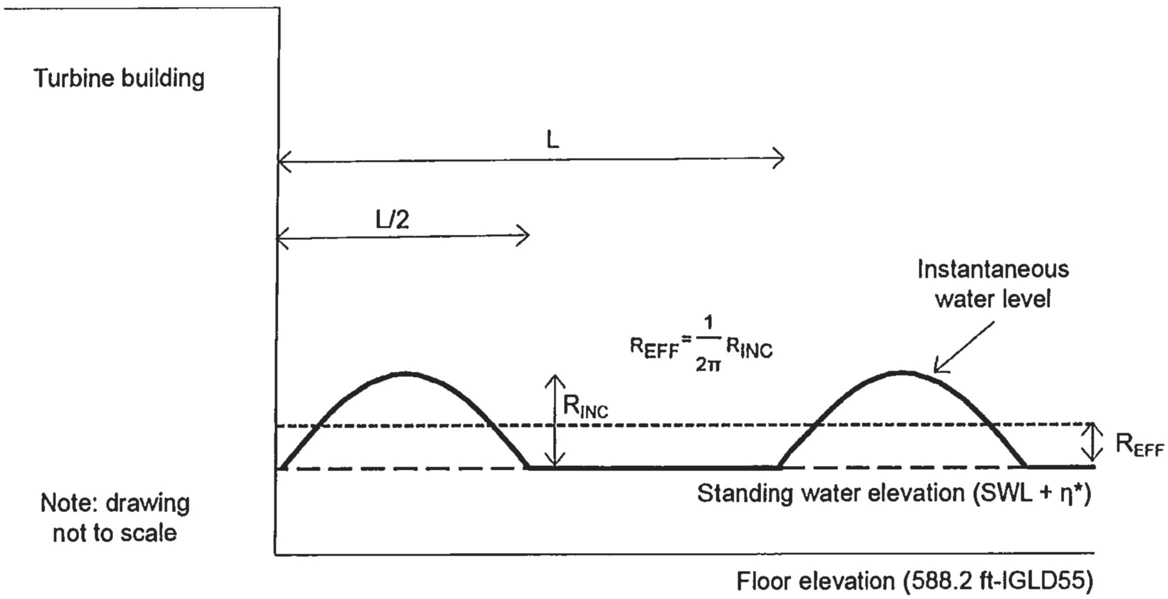
$$R_{EFF\ 50\%} = \frac{0.88}{1.86} R_{EFF\ 2\%} \qquad (6\text{-}4)$$

where:

$R_{EFF\ 50\%}$ = effective runup height proxy for $\bar{R}$ (ft)
$R_{EFF\ 2\%}$ = effective runup height proxy for $R_{INC}$ (ft)

The coefficients in Equation 6-4 (0.88 and 1.86) are found in Section II-4-4 of USACE (2011). This value ($R_{EFF\ 50\%}$) can be used as a proxy for average runup elevation, although it should be noted that this elevation is not static and the instantaneous level oscillates around this value, as shown in Figure 6-1. Physically, $R_{EFF\ 50\%}$ is equivalent to the average height that runup will reach 50% of the time. Similarly, $R_{EFF\ 2\%}$ is the average height that runup will reach 2% of the time.

Thus, SWL+$\bar{\eta}$+$R_{EFF\ 50\%}$ ($WL_{50\%\ EFF}$) where $\bar{\eta}$ is the mean wave setup, can be thought of as a proxy for the mean water level. Instantaneous water elevations range between the standing water level and the maximum extent of runup. $WL_{50\%\ EFF}$ is used to compute mean water leakage into the turbine building.




* Note: wave setup (η) was considered either as a maximum modeled level ($\eta_{MAX}$), which was used in calculations for the structural loading analysis, or a mean level ($\bar{\eta}$), which was used in the computations for average external water level proxies at the turbine building.

**Figure 6-1: Effective Runup Schematic**

## 7. Calculations

These cases examine the incident wave runup for the largest (significant wave height, $H_S = H_0 = 23.5$ feet) and longest (T = 10 seconds) deep-water waves with no barriers present and each evaluated SWL, shown in Table 7-1 (ENERCON, 2013a). From Equation D.4.5-10, the deep-water wavelength ($L_0$) of these waves would be 512.32 feet. These parameters provide a worst-case flooding scenario for each initial SWL; all input values are shown in Table 5-1. For these deep-water wave conditions, the maximum wave setup ($\eta_{MAX}$) was 1.74 feet, corresponding to the still water level of 587 ft-IGLD55, as found in FPL-076-CALC-003 (ENERCON, 2013a). For the lower still water levels, the maximum wave setup narrowly ranged between 1.20 and 1.28 feet (ENERCON, 2013a). This result is likely attributable to the relatively steep sloping bathymetry at the seaward edge of PBNP. For the highest still water level, 587.64 ft-IGLD55, the maximum setup at BP2 was 1.46 feet. Above a threshold water level (between 586 and 587 ft-IGLD55), waves and accompanying setup make it close to the turbine building; below it, the wave setup height appears to be quite stable. Please refer to FPL-076-CALC-003 for a more comprehensive discussion and presentation of the DELFT3D setup results.

Mean wave setup ($\bar{\eta}$) ranged from 1.12 to 1.39 feet (ENERCON, 2013a). If observation point BP2 was submerged, then it was used to determine the maximum and average wave setup, as it was directly adjacent to the turbine building (see Figure 2-1). If that point was dry for the entire simulation, then observation point SS4 or SS5, the next closest inundated points, were used. The mean wave setup was computed for only the durations in which BP2 (or neighbor observation point) experienced flooding, which often lagged the start of the model by several minutes (ENERCON, 2013a).

The largest waves, responsible for the highest incident runup elevations, break furthest seaward of the turbine building (Figure 7-1). The slope of the beach/beachface for this calculation was divided into two segments, after the methodology shown in Figure D.2.8-1 of FEMA (2007). The landward segment extended from near the turbine building (observation point BP2, see Figure 2-1) out to the edge of the topographic map of PBN. The elevation of the BP2 is 587.35 feet-IGLD55 and the elevation at the seaward edge of the topographic map is 573.35 feet-IGLD55. The distance between these points is 350 feet, providing an upper beach slope of 0.04. Seaward of this segment, the beach slope is 0.01 out to 1,000 feet from the turbine building (S&L, 1976). The DELFT3D model showed wave breaking initiating ~700 feet from the turbine building (Figure 7-1), so the horizontal extent of this segment is 350 feet. A weighted average of the overall beach slope was computed to yield an average value of 0.025. This beach slope (m) was used in all of the wave runup calculations, since only minor shifts (~20 feet) occurred in wave breaking locations due to different initial still water levels.

From the calculated beachface slopes and deep-water wave conditions, the incident wave runup ($R_{INC}$) was calculated to be 1.65 feet. This oscillatory swash operates on top of the wave setup ($\eta$), so this result must be added to the SWL (583, 585, 586, 587, OR 587.64 ft-IGLD55) and modeled maximum or mean setup. Thus, the water elevation ($SWL+\eta_{MAX}+R_{INC\ 2\%}$) which would be exceeded less than 2% of the storm is 590.39 ft-IGLD55 in the 587 ft-IGLD55 SWL case, the most severe still water level in the IPEEE (PBNP, 1995; PBNP, 2013). For this incident wave runup, an instantaneous water depth of up to 2.19 feet at 590.39 ft-IGLD55 (Table 7-2) when SWL = 587 ft-IGLD55 could be realized at the turbine building. This water depth may be used to calculate loading against the turbine doors. Physically, this elevation is a close proxy for the maximum water elevation for the provided still water level, wind, and wave characteristics, but could be exceeded less than 2% of the storm. The effective runup ($R_{EFF\ 2\%}$), calculated from Equation 6-3, is 0.26 feet. The effective height that 50% of individual wave runups will exceed ($R_{EFF\ 50\%}$), calculated from Equation 6-4, is 0.12 feet.

Using inputs from Table 5-1 and the calculated $R_{INC}$, $R_{EFF\ 2\%}$, and $R_{EFF\ 50\%}$ from section 6.2 and 6.3, water level proxies were computed for turbine building's east wall. Results are shown in Table 7-3, and the most severe IPEEE case is shown schematically in Figure 7-2 (PBNP, 1995; PBNP, 2013). The incident runup remains the same in each case, but the overall water elevation proxies differ (Table 7-3). When the BP2 observation point was dry, the next closest submerged observation point (SS4 or SS5) was used to determine the maximum or mean wave setup (see Figure 2-1 and ENERCON, 2013a). The mean water surface proxy ($WL_{50\%\ EFF}$) is above the elevation of the turbine building floor during only the most elevated initial still water levels, 587 and 587.64 ft-IGLD55. For the 586 ft-IGLD55 case, runup would be expected to impact the structure less than 0.65 feet (588.85 ft-IGLD55) for 98% of the storm. The effective runup elevations indicate no permanent ponding due to wave setup and swash. In the other two initial water level cases, 583 and 585 ft-IGLD55, more than 98% of the incident wave runups would not reach the turbine building floor; similarly, the mean water surface elevation proxy indicates no persistent water on the building doors.

A maximum mean setup ($\bar{\eta}$) also was used to calculate the total effective water elevation proxies used to determine leakage into the turbine building. The mean water level proxy ($WL_{50\%\ EFF}$) is above the turbine building floor elevation by 0.31 feet in the most severe IPEEE case, SWL = 587.00 ft-IGLD55 (PBNP, 1995; PBNP, 2013). No standing water is predicted for the SWL = 586 ft-IGLD55 case, with $WL_{50\%\ EFF}$ = 587.24 ft-IGLD55. Further, no standing water and less than 2% of incident wave runup is expected for the initial SWL = 583 or 585 ft-IGLD55 cases.

**Table 7-2: Calculation Results for the Maximum Structural Loading Analysis. Inputs for Maximum Incident Runup ($R_{INC\ 2\%}$) and Setup ($\eta_{MAX}$) for each SWL Case. Output is a Dynamic Total Water Level ($WL_{2\%}$).**

| $H_0$ (ft) | T (s) | m (ft/ft) | $R_{INC\ 2\%}$ (ft) | SWL (ft-IGLD55) | Frequency of SWL ($yr^{-1}$)* | $\eta_{MAX}$ (ft) ** | SWL + $\eta_{MAX}$ (ft-IGLD55) | $WL_{2\%}$ (ft-IGLD55) |
|---|---|---|---|---|---|---|---|---|
| 23.50 | 10.00 | 0.025 | 1.65 | 587.64 | 9.9E-07 | 1.46 | 589.10 | 590.75 |
| 23.50 | 10.00 | 0.025 | 1.65 | 587.00 | 4.2E-06 | 1.74 | 588.74 | 590.39 |
| 23.50 | 10.00 | 0.025 | 1.65 | 586.00 | 5.4E-05 | 1.20 | 587.20 | 588.85 |
| 23.50 | 10.00 | 0.025 | 1.65 | 585.00 | 3.2E-04 | 1.21 | 586.21 | 587.86 |
| 23.50 | 10.00 | 0.025 | 1.65 | 583.00 | 1.4E-02 | 1.28 | 584.28 | 585.93 |

* SWL frequencies are provided by PBNP (2013), formulated from data in PBNP (1995).
** maximum setup values ($\eta_{MAX}$), measured at either observation point BP2 or SS5 (if BP2 was dry), obtained from DELFT3D simulations (FPL-076-CALC-003) corresponding to still water levels (SWL) and deep-water wave conditions of $H_0$ = 23.5 feet and direction = 120 °.

Note: This table represents a temporary condition that would induce maximum wave runup for use in calculating structural loading on the turbine building.

**Table 7-3: Calculation Results for the Average External Water Level Proxies ($WL_{50\%\ EFF}$) on the Turbine Building. Inputs for Highest Effective Incident Runup ($R_{EFF\ 50\%}$) and Mean Setup ($\bar{\eta}$) for each SWL Case.**

| $H_0$ (ft) | T (s) | m (ft/ft) | $R_{EFF\ 50\%}$ (ft) | SWL (ft-IGLD55) | Frequency of SWL ($yr^{-1}$) * | $\bar{\eta}$ (ft) ** | SWL + $\bar{\eta}$ (ft-IGLD55) | $WL_{50\%\ EFF}$ (ft-IGLD55) |
|---|---|---|---|---|---|---|---|---|
| 23.50 | 10.00 | 0.025 | 0.12 | 587.64 | 9.9E-07 | 1.13 | 588.77 | 588.89 |
| 23.50 | 10.00 | 0.025 | 0.12 | 587.00 | 4.2E-06 | 1.39 | 588.39 | 588.51 |
| 23.50 | 10.00 | 0.025 | 0.12 | 586.00 | 5.4E-05 | 1.12 | 587.12 | 587.24 |
| 23.50 | 10.00 | 0.025 | 0.12 | 585.00 | 3.2E-04 | 0.72 | 585.72 | 585.84 |
| 23.50 | 10.00 | 0.025 | 0.12 | 583.00 | 1.4E-02 | 0.84 | 583.84 | 583.96 |

* SWL frequencies are provided by PBNP (2013), formulated from data in PBNP (1995).
** mean setup values ($\bar{\eta}$) averaged over the entire simulation when flooding occurred, measured at either observation point BP2 or SS4 (if BP2 was dry), obtained from DELFT3D simulations (FPL-076-CALC-003) corresponding to still water levels (SWL) and deep-water wave conditions of $H_0$ = 23.5 feet and direction = 120 °.

Note: This table represents average water level proxies at the turbine building.

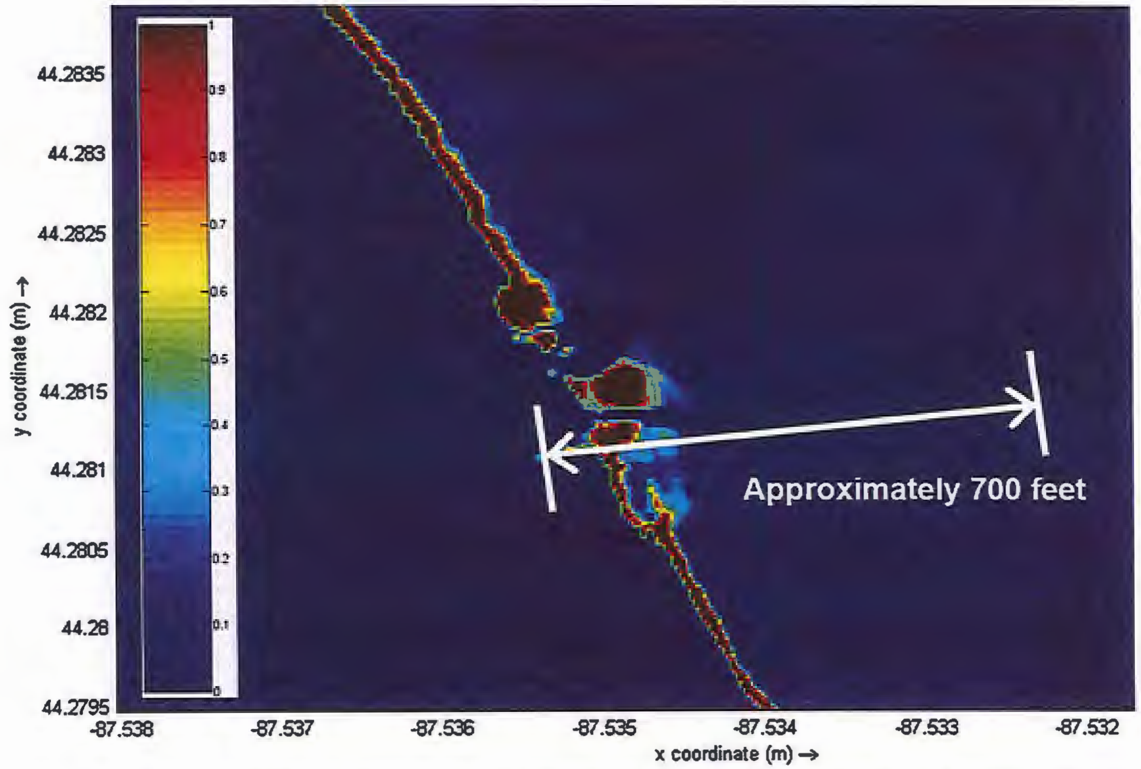
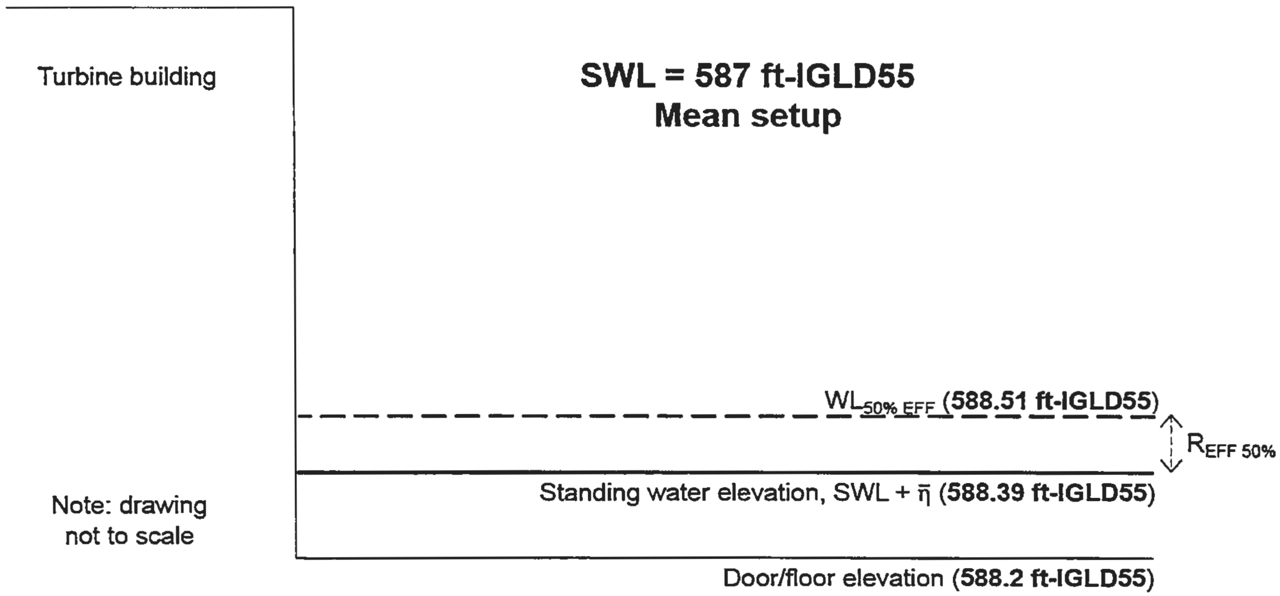



Figure 7-1: Maximum Wave Breaking Distance of the Largest Waves Normal to the Southwest Corner of the Pump House/Turbine Building at Point Beach Nuclear Plant (PBNP). Percent Breaking (0%=No Breaking, 1=100% Breaking) is Shown in the Color Map Above.




Results for deep-water wave height = 23.5 ft, deep-water wave direction = 120 °, SWL = 587 ft-IGLD55, and no barriers present.

Legend

$R_{INC}$ = max. incident wave runup
$R_{EFF}$ = effective wave runup ('time-averaged' $R_{INC}$)
SWL = still water level (587 ft-IGLD55)
$\bar{\eta}$ = mean wave setup at BP2 (ft)
$WL_{50\%\ EFF}$ = 'average' water elevation proxy including $R_{EFF\ 50\%}$

**Figure 7-2: Schematic of Results from Table 7-3 for SWL=587.00 ft-IGLD55 using the Mean Wave Setup. The $WL_{2\%\ EFF}$ and $WL_{50\%\ EFF}WL_{2\%}$ are the Effective Water Level Proxies Used to Determine Water Leakage into the Turbine Building.**

# Attachment A (On DVD)

| File Name (References) | Revision | Pages/ Worksheets/ File |
|---|---|---|
| Dean and Dalrymple, 2007 | NA | File |
| Delatres, 2012.pdf | NA | File |
| ENERCON, 2013a | 0 | File |
| ENERCON, 2013b | 0 | File |
| FEMA, 2005 | NA | File |
| FEMA, 2007 | NA | File |
| FEMA, 2012 | NA | File |
| Mase, 1988 | NA | File |
| Melby, 2012 | NA | File |
| PBNP, 1995 | NA | File |
| PBNP, 2013 | NA | File |
| S&L, 1967 | NA | File |
| Stockdon, 2006 | NA | File |
| USACE, 2011.pdf | NA | File |
| USACE, 2012.pdf | NA | File |

**ATTACHMENT 2**

**NEXTERA ENERGY POINT BEACH, LLC**
**POINT BEACH NUCLEAR PLANT**

**POINT BEACH REVISED EXTERNAL FLOOD SAFETY SIGNIFICANCE DETERMINATION**

**Performance Deficiency**

The licensee failed to establish appropriate procedural requirements to implement external flooding wave run-up protection design features as described in the FSAR.

**Executive Conclusion**

The safety significance of this issue is assessed to be of very low with margin for Units 1 and 2. Table 4 provides the core damage frequency (CDF) with and without barriers as well as the change in core damage frequency with and without the barriers. The basis of this conclusion is that a detailed wave run-up analysis results in a calculated water level much lower than the water levels previously evaluated in the IPEEE. The PRA analysis based on the updated engineering analysis confirms that the IPEEE water levels were dominated by estimates and assumptions that resulted in excessive conservatism for the 4.2E-06/yr frequency event. These lower water levels from the updated engineering analysis result in fewer equipment impacts.

**Background**

The IPEEE response to GL 88-20, "Individual Plant Examination for Severe Accident Vulnerabilities," evaluated external flood hazards for PBNP. This evaluation was based in part on the analysis for external flood events conducted in conjunction with the NRC's TAP A-45 study. In order to evaluate the safety significance of this issue, the data provided in the IPEEE was used to evaluate the change in CDF and large early release frequency (LERF). For the purpose of this evaluation the "change" being considered is the plant with and without the barrier protection to 589.2 (IGLD-1955), as described in the IPEEE report.

To perform this evaluation, some simplifying conservative assumptions, providing margin, are made:

1) Above 589.2 ft. IGLD 1955 (+9 ft.), the impact of the flood is the same with and without barriers.
2) For the purposes of the PRA calculations, the water level inside the buildings was assumed to equal the water level outside the Turbine Building at time zero.
3) Below 588.2 ft. IGLD 1955 (+8 ft.), there is no impact from the flood (with or without barriers).
4) The PRA evaluation assumes a concurrent dual unit loss of offsite power (LOOP) due to the storm which is conservative because the postulated storm does not reach sustained wind speeds that are expected to cause damage to offsite power distribution.

## Risk Assessment

PBP PRA Model Rev. 5.02 was used for this assessment.

Since this evaluation will be applying the frequency of the external flood outside of the PRA model, all initiators in the internal events model were set to 0.0 with the exception of the weather-centered LOOP initiator (INIT-T1W). By doing this, the value being quantified is the conditional core damage probability (CCDP), i.e., the core damage probability assuming the initiator (external flood in this case) occurs

The following steps were taken to evaluate the significance of this issue:

1) Run CAFTA cases (average Testing & Maintenance) for Units 1 and 2 with an E-10 truncation limit with flags set to account for the postulated equipment failures. The results of the cases representing the CCDPs for the five bins comprising varying depths of water on the Turbine Building floor are shown in Table 1. For simplicity, only the maximum CCDP for each bin will be carried through the rest of this calculation.

2) In order to calculate ΔCDF for the water height ranges in this report, flood event frequencies had to be derived. That was done by defining a relationship between calculated water levels and still water levels from Attachment 1. This relationship is shown in Table 2.

3) The results of the curve-fit of the flood exceedance frequencies from Table 5.2.5-2 of the IPEEE are presented in Table 3. Note that due to the data, two curve fits are presented. The first curve fit represents still water elevations ≤585.1 ft IGLD 1955 and the second curve fit represents still water elevations > 585.1 ft IGLD 1955.

4) By using these developed relationships, a frequency is derived for a given calculated flood level by determining the associated still water level from Table 2 and then using it to determine the frequency from Table 3.

5) The CDF is calculated by multiplying the Incremental Flood Frequency (determined from the relationships in Tables 2 and 3, by the CCDP and is presented in Table 4.

6) The ΔCDF was calculated by subtracting the CDF with barriers from the CDF without barriers for each bin. The Total ΔCDF was obtained by subtracting the CDF Total with barriers from the CDF Total without barriers. This information is also presented in Table 4.

Note that based upon previous evaluations and the very small CDF values, values for LERF were not calculated. Due to the nature of the initiating event, it is judged that there is no unique challenge to LERF. Thus, the ΔLERF for this evaluation is judged to be well below 1E-09 /yr.

The final calculation of ΔCDF for this issue is determined to be 7.E-09 /yr, which is of very low safety significance, with margin.

## Margin

The flood consequence is considered to be bounding and conservative for the following reasons:

1) All equipment affected by the flood is assumed to be failed at time zero. Based on engineering evaluations, the water accumulation inside various areas of the plant would take over three hours prior to affecting any safety significant equipment.
2) No credit for flood mitigation actions taken in response to rising water levels throughout the plant has been modeled. Due to the relatively slow progression of the postulated flood, there should be time for the operators or plant staff to respond to the rising water level and to protect and/or realign equipment.
3) No credit for recovery actions taken in response to equipment issues in the plant has been modeled. It is expected that some equipment may be able to be recovered and that other means to provide decay heat removal could be used, e.g., pumper trucks, B.5.b equipment, and portable generators.
4) The concrete barriers installed at a lake level of 580.7 ft. (IGLD 1955), in accordance with PC 80 Part 7, "Lake Water Level Determination," are assumed to be ineffective in limiting the quantity of water.

## Table 1

Maximum Conditional Core Damage Probability vs. Water Level Range Bins

| Bin | Range of Water Level on Turbine Building Floor (inches) (2,3) | Equipment Assumed Failed (1,7) | CCDP (max) (4) |
|---|---|---|---|
| 1 | 0 to <4.0 | Offsite power assumed lost,<br>Offsite Power Transformers (1X-01/03, 2X-01/03),<br>RHR Pumps (1/2P-10A/B),<br>RHR Pump Suction from Containment Sump B (1/2SI-851A/B) | **4.25E-05** |
| 2 | 4.0 to <8.0 | Charging Pumps (1CV-2A/B/C and 2CV-2A/B/C),<br>Station Battery Chargers (D-07/D-08/D-09) | **6.87E-04** |
| 3 (5) | 8.0 to <12.5 | A Train Emergency Diesel Generators (G-01, G-02),<br>G-01/G-02 EDG Alarm & Electrical Panels (C-34/C-35),<br>G-01/G-02 EDG DC Power Transfer Control Panels (C-78/C-79),<br>4.16 KV Switchgear (1/2A-03/04),<br>4.16 KV Vital Switchgear A Train (1/2A-05),<br>1/2HX-11A, B RHR HX Shell Side Inlet Valves (1/2CC-738A/B),<br>Non-Safety Related 480V MCCs (B-33, B-43),<br>Steam Generator Feedwater Pump Seal Water Injection Pumps (1/2P-99A/B) | **7.70E-03** |
| 4 (6) | 12.5 to <17 | 480 V Vital MCCs A Train (1/2B-32),<br>Safeguards Batteries (D-01, D-02),<br>Service Air Compressor (K-3B,)<br>Diesel Driven Fire Pump (P-35B),<br>Instrument Air Compressors (K-2A/B) | **8.92E-02** |
| 5 | $\geq$17 | Condensate Pumps (1/2P-25A/B),<br>Feedwater Pumps (1/2P-28A/B),<br>Service Water Pumps (P-32A/B/C/D/E/F),<br>DC Distribution Panels (D-63, D-64),<br>Stand-by Steam Generator Pumps (P-38A/B),<br>Turbine Driven Auxiliary Feedwater Pumps (1/2P-29),<br>Motor Driven Auxiliary Feedwater Pumps (1/2P-53),<br>Service Air Compressor (K-3A),<br>Safety Injection Pumps (1/2-P12A/B) | **1.00** |

Notes:

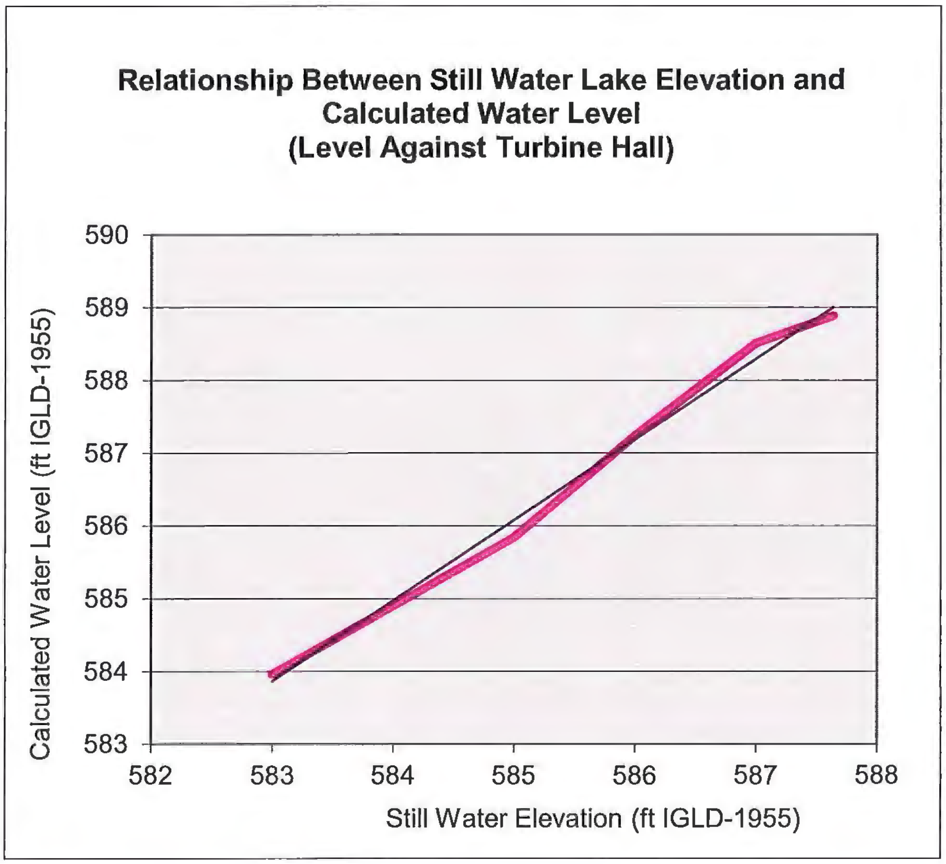
(1) "Equipment Assumed Failed" for each range of water levels greater than 588.2 ft. IGLD 1955 is based on the elevation of the limiting vulnerable subcomponent.
(2) "Range of Water Level" is based on inches of water on the turbine building floor.
(3) "Range of Water Level" 0 inches equals 588.2 ft. IGLD 1955.
(4) The maximum CCDP from either unit is used in the downstream calculations.
(5) It has been identified that P-35A, Electric Fire Pump, may fail at an elevation in Bin 3. Since this bin already fails 1A-05 (which powers 1B-03, which powers the electric fire pump), there is no additional consequence of this component failure.
(6) It has been identified that a control panel associated with the 2P-29, Turbine Driven Aux Feedwater Pump low suction pressure trip, may fail at an elevation in Bin 4. A sensitivity case was run that showed that the CCDP value would increase slightly to 9.0E-2. This small difference compared to the CCDP value used for Bin 4 is not significant in the conclusions of this evaluation.
(7) Equipment failures at the water level elevations have been validated against the most recent walk-downs as documented in EC 279398.

## Table 2

Calculated Water Level Based on Still Water Lake Elevation

### Still Water Elevation to Calculated Water Level Relationship

| Still Water Lake Elevation (ft. IGLD-1955) | Calculated Water Level (ft. IGLD-1955) * |
|---|---|
| 587.64 | 588.89 |
| 587.00 | 588.51 |
| 586.00 | 587.24 |
| 585.00 | 585.84 |
| 583.00 | 583.96 |




NOTE:
* Calculated Water Level is taken from Table 7-3 of Enercon Calculation FPL-076-CALC-004

**Table 3**

Annual Frequency Based on Still Water Elevation
[Derived from Information in IPEEE]

**Flood Frequency - Curve Fit Equations**

| Annual Frequency (per yr) | Still Water Elevation (ft IGLD 1955) from IPEEE Table 5.2.5-2 | *Frequency - Curve Fit (per yr)* IPEEE StillWaterFREQ1 | IPEEE StillWaterFREQ2 |
|---|---|---|---|
| 3.69E-02 | 582.5 | 3.7E-02 | |
| 2.53E-04 | 585.1 | 2.7E-04 | 4.8E-04 |
| 3.45E-07 | 588.0 | | 4.1E-07 |
| 8.25E-11 | 591.0 | | 2.9E-10 |

Note where two values are provided, IPEEE StillWater FREQ2 was used.

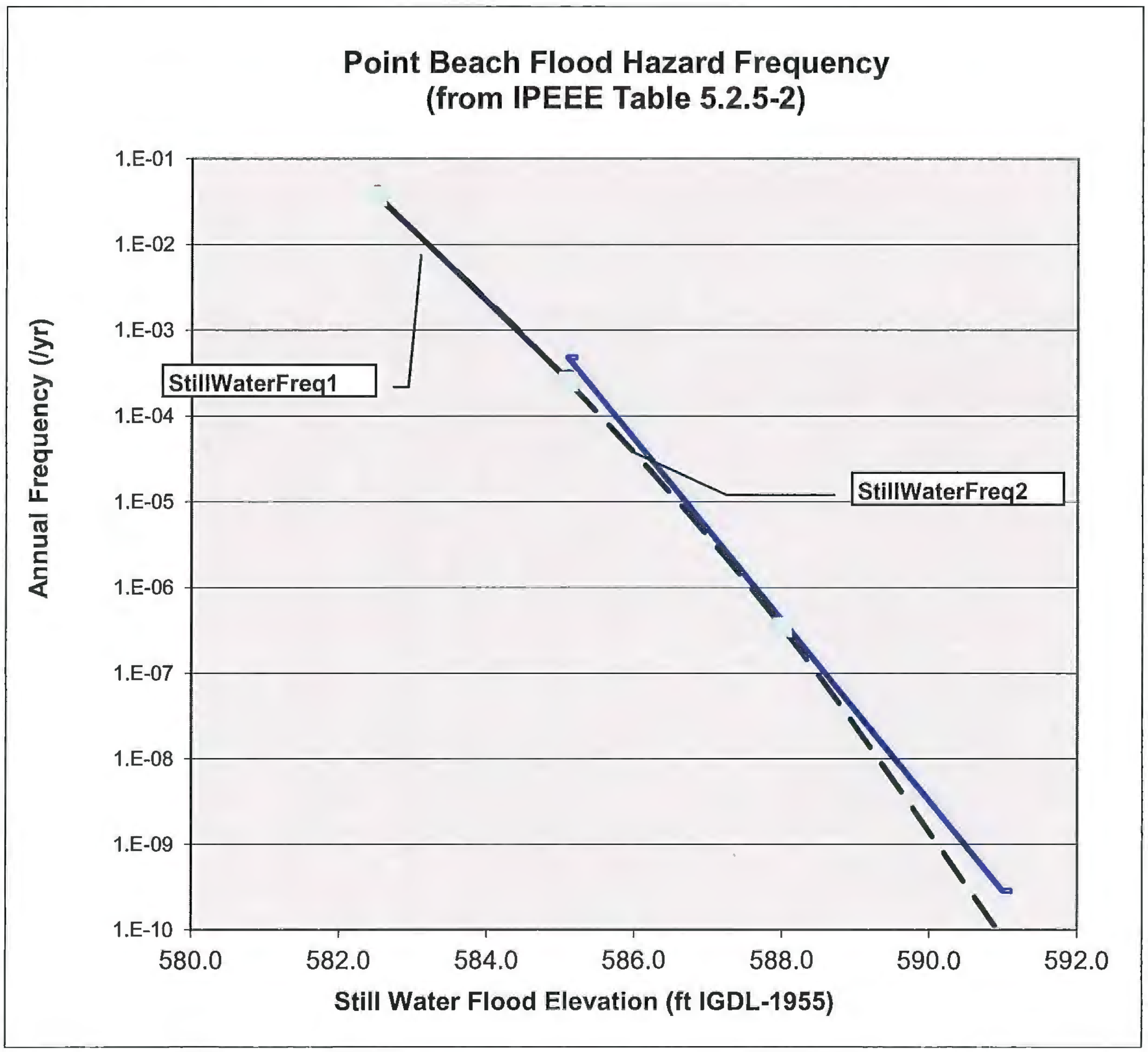


**Table 4**

ΔCDF Calculation With and Without Barriers

| Bin | Flood Frequency | Incremental Flood Frequency | Still Water Lake Elevation | Effective Water Level (Range) (1) | Without Barriers CCDP | Without Barriers CDF | With Barriers CCDP | With Barriers CDF | $\Delta CDF_i$ |
|---|---|---|---|---|---|---|---|---|---|
| | *per yr* | *per yr* | *ft (IGLD-1955)* | *inches* | | *per yr* | | *per yr* | *per yr* |
| 1 | 5.6E-06 | 2.9E-06 | 586.93 | 0 to <4.0 | 4.25E-05 | 1.23E-10 | 0.00E+00 | 0.00E+00 | 1.23E-10 |
| 2 | 2.7E-06 | 1.4E-06 | 587.23 | 4.0 to <8.0 | 6.87E-04 | 9.54E-10 | 0.00E+00 | 0.00E+00 | 9.54E-10 |
| 3 | 1.3E-06 | 7.3E-07 | 587.53 | 8.0 to <12.5 | 7.70E-03 | 5.60E-09 | 0.00E+00 | 0.00E+00 | 5.60E-09 |
| 4 | 5.7E-07 | 3.2E-07 | 587.87 | 12.5 to <17 | 8.92E-02 | 2.84E-08 | 8.92E-02 | 2.84E-08 | 0.00E+00 |
| 5 | 2.5E-07 | 2.5E-07 | 588.21 | >17 | 1.00E+00 | 2.48E-07 | 1.00E+00 | 2.48E-07 | 0.00E+00 |
| | | | | | **CDF Total** | **2.83E-07** | **CDF Total** | **2.76E-07** | **7.E-09** |

(1) Effective water level is the level of water in the turbine building.

## ATTACHMENT 3

## NEXTERA ENERGY POINT BEACH, LLC
## POINT BEACH NUCLEAR PLANT

## CONTINUED SERVICE WATER PUMP AVAILABILITY WITH LOSS OF DC CONTROL POWER

As discussed in Reference 2, a significant contributor to the difference between initial NextEra and NRC staff estimated changes in Core Damage Frequency (CDF) with failure of equipment up to 589.2 feet of water IGLD 1955 is the PRA modeling of Service Water (SW) pump availability upon loss of DC control power. NRC's SPAR model contains a presumption that the SW pump becomes unavailable upon loss of DC power. (It is important to note that NextEra's updated analysis does not result in water levels which would impact DC power.)

With a Loss of Offsite Power (LOOP), all four installed Emergency Diesel Generators (EDGs) will start and energize their associated buses. All six Service Water (SW) pump supply breakers will then sequence onto their respective AC load centers.

With respect NRC's SPAR model assumption that SW would be unavailable upon loss of DC power, an interruption in DC control power does not cause re-positioning of breakers. DC control power provides remote breaker operation of the 480 volt SW supply breakers by momentarily energizing either the opening or closing solenoid coils. The overcurrent protective device is not dependent on DC control power and remains functional without DC control power. Once a breaker is closed, a loss of DC control power will not cause it to open, and the connected load will continue to be energized. Therefore, the operating SW pumps will remain in operation if DC control power is lost.

DC control power supplies the starting circuit of the EDGs, the power to initially flash the field on an EDG that is starting, and provides the ability to remotely adjust the electric governor setpoint, and to adjust the voltage regulator setpoint. If an operating EDG suffers a complete loss of DC control power, the electronic governor will fail to full fuel demand, and the backup mechanical governor will take over speed regulation. The exciter and voltage regulator are self-energized from the generator output, and will fail to the as-set voltage. The ability to locally adjust frequency on the running generator will remain available.

The pending loss of DC control power due to flooding would be anticipated, and would be acted upon by the Operator before the actual loss occurred. Each battery charger would initiate a trouble alarm when it failed, providing a minimum of 1 hour notice of the loss of the supplied bus. The one hour time is based on the minimum capacity of the batteries, and provides time to re-align the DC control power supplied to running EDGs, 4 kV switchgear, and 480 V Load Centers prior to the complete loss of control power.

Abnormal Operating Procedure AOP-0.0 directs the Operator to realign control power from the normal source to the alternate DC control power source. This includes realigning the DC control power for Emergency Diesel Generators and 4 kV switchgear, and the direction would be exercised in anticipation of the loss of the busses when the inability to recover the overheated chargers became evident.

If a running "B" train EDG were lost for any reason, the remaining "B" train EDG would be aligned to re-energize the bus previously supplied by the lost EDG using either ECA-0.0 (loss of AC power) or OI-35A (standby emergency power alignment). This would ensure the availability of power to all 3 "B" train SW pumps.

In the event that a SW pump needs to be started on an energized bus, but does not have DC control power available to effect remote operation, the procedural direction to "start pumps as necessary" will cause the Operator to start the pump by closing the breaker locally. The low service water pressure alarm response procedure would direct this action, as would direction in the various abnormal operating and emergency operating procedures.

Based on these considerations, it is concluded that the postulated combination of a LOOP, loss of the "A" train buses, and loss of the D-07, D-08, D-09 battery chargers does not limit the plant to only a single SW pump during long term operation.

**The actions for switching to alternate DC Control power are as follows:**

- The operators will respond to control room alarms based on degrading DC voltage
- AOP 0.0 Vital DC System Malfunction, entry conditions would be satisfied and direct switching to alternate DC Control power for G-04 EDG.
- If G-04 EDG would lose DC control power, guidance for local speed control is provided in ECA 0.0, Loss of all AC, for hydraulic governor operation.
- AOP 10A, Safe Shutdown local control procedure, provides guidance for local operation of the Service Water pump breakers, if local operation, is required.
- A Nuclear Oversight Observer completed a satisfactory observation of the above actions in the field by 3 Auxiliary Operators.
- The above field actions were timed and validated to be completed in aggregate of <1 hour.
- These Operator actions are part of the INPO accredited training programs for both initial and continuing training for the Auxiliary Operators. Local operation of these breakers is trained on, tested, and evaluated by our Operator Initial and Continuing Training programs.

**Conclusion**

If a Service Water Pump is operating when DC Control Power is lost it will continue operating and if it is not operating when DC Control Power is lost, the Operators are trained and are procedurally directed to start the pumps locally. If DC Control Power is lost to G-04 EDG, the operators are trained and have procedural guidance to locally control G-04 EDG's speed with the hydraulic governor.

ATTACHMENT 4

NEXTERA ENERGY POINT BEACH, LLC
POINT BEACH NUCLEAR PLANT

# PBNP TIME TO RESPOND TO RATES OF RISE IN LAKE MICHIGAN WATER LEVEL

## Purpose

The purpose of this evaluation is to establish the time available to respond to rising levels in Lake Michigan before the design basis flood threat may be reached. This evaluation does not rigorously reevaluate the point at which there is a threat from rising water.

## Design and Licensing Basis

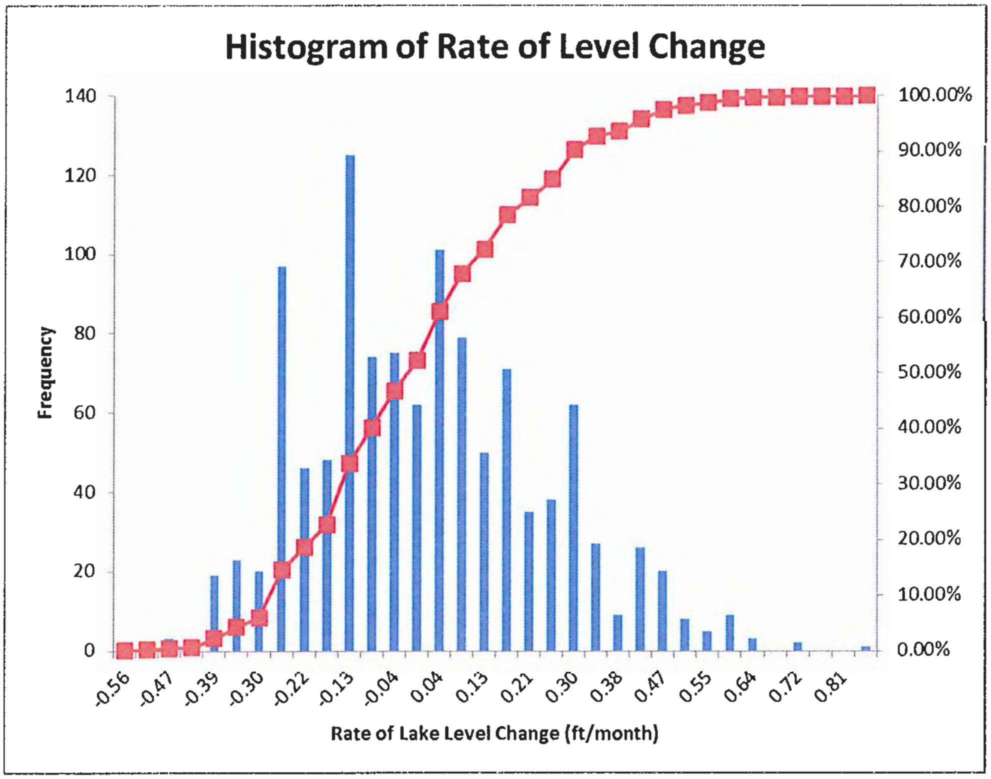
No flood height elevation has been calculated for anything but the vertical wall on the east side of the forebay to date. This value is stated in the FSAR (Section 2.5) as 8.42 feet plant elevation. It is based on a maximum undisturbed lake level of +1.7 ft. plant elevation plus a wave run-up of 6.55 feet against a vertical surface, and a sustained level change of +0.17 feet of water based on conservative value of sustained easterly wind velocity of 40 mph over a fetch length of 70 miles and average depth of 465 feet of water. Thus, the maximum expected run-up on a vertical structure would be 6.72 feet above the normal water level (resulting in a plant elevation of 8.42 feet) and somewhat less on a riprap slope.

## Plant Reaction to Lake Level Changes

Prior to Revision 4 (issued March 14, 2013) Point Beach procedure PC 80 Part 7 required installation of pre-cast concrete barriers within 3 weeks when the level of Lake Michigan reached a reported level of 580.7ft. IGLD 1955. The detailed directions in the procedure on how to obtain this information would have resulted in using the currently accepted International Great Lakes Datum ("IGLD") of 1985. This elevation equates to -0.2 feet plant elevation. This procedure is performed monthly and ensures advance preparation in anticipation of a potential high water event.

## Historical Lake Level Changes

To determine how much advance notice it would have ensured, the historical lake data archived by the National Oceanographic and Atmospheric Administration (NOAA) was reviewed.
The data for monthly average lake level contained data from January 1918 through March 2013 was converted to feet, and the difference between successive months calculated to obtain the monthly rate of level change. As shown in the histogram below, the distribution of level change is asymmetrical.




Because the lake level as a function of time cannot be replicated by any meaningful function, it is instructive to calculate the maximum historical one month and two month level changes. On one occasion, the rate of rise in the lake reached 0.85 ft in a single month (April, 1960). The next highest rate of rise ever observed was 0.69 ft/month during two successive months (April and May 1929). As the length of the period increases, the monthly average rate of rise decreases. The maximum monthly rate of rise for a three month period is 0.55 ft/month, and for a four month period it has been 0.48 ft/month. A full listing of the data is appended to this evaluation.

## IPEEE Trigger Points

From Table 3-3 in the TAP A-45 report (recreated in the IPEEE submittal), the combination of lake level and wave run-up which gets to the 8ft plant elevation (588.2ft IGLD 1955) occurs with a still lake level of about 582ft IGLD 1955. Since 580.2 IGLD 1955 corresponds to 0.0 feet plant elevation, the still lake level at which the wave run-up reaches 8.0 feet, is 1.8 feet plant elevation. Using this as the lake level at which the threat from rising water materializes, there is 2 feet between the "install barriers" trigger point contained in PC 80 Part 7 and the threat level.
Using the maximum historical rise rates for one, two, three, and four months to consume the entire 2 foot margin (height differential) would require a period (denoted as "available time period") of:

$$T = \left(\frac{2\,ft}{Rate\ (n\ Months)}\right) \cdot 4.33\,\frac{weeks}{month}$$

Using this formulation, the following results are obtained:

| Height Differential (ft) | Rate of Rise | Rate Period | | Available Time Period |
|---|---|---|---|---|
| | ft/month | months | weeks | From Trigger Point (Weeks) |
| 2 | 0.48 | 4 | 17.3 | 18.1 |
| 2 | 0.55 | 3 | 13.0 | 15.8 |
| 2 | 0.69 | 2 | 8.7 | 12.6 |
| 2 | 0.85 | 1 | 4.3 | 10.2 |

In all cases, the available time period for the 2 foot rise exceeds the highest historical value for that same time period. Therefore, the rate of rise and the available time for each case is conservative.

This indicates that even in the "worst case" where lake level were rising at the most rapid historic 1 month rate of 0.85 ft/month, and sustained it for an unprecedented 10 weeks, it would still take a little more than those 10 weeks to consume the 2 feet of margin from the time that the trigger point is reached until the threat level was reached.

However, the surveillance is only performed monthly. So it is possible that the reported lake level could be just below (e.g., 0.1 foot less than) the "trigger point" level of 580.7 ft at the time that the procedure is performed. It would then take another month (4.3 weeks) to discover that the trigger point level had been exceeded. Under this postulation, it is appropriate to use the rise rate for n+1 months to determine time to achieve the total rise.

$$T = \left[\left(\frac{2\,ft}{Rate\ (n+1\ Months)}\right) - 1\ month\right] \cdot 4.33\,\frac{weeks}{month}$$

Using this formulation, the following results are obtained:

| Height Differential (ft) | Rate of Rise | Rate Period | | Available Time Period |
|---|---|---|---|---|
| | ft/month | months | weeks | From Detection to Threat Level (Weeks) |
| 2 | 0.48 | 4 | 17.3 | - |
| 2 | 0.55 | 3 | 13.0 | 13.7 |
| 2 | 0.69 | 2 | 8.7 | 11.4 |
| 2 | 0.85 | 1 | 4.3 | 8.2 |

Using the conservative approach described above, it would still leave at least 8.2 weeks, after discovery that the trigger point had been exceeded, to complete preparations for high water, even if the first opportunity had been missed. The procedure allows three weeks for installation so that there would be 5.2 weeks available to address barrier deficiencies before lake level reaches the design basis flood level. At the time that the barriers would have been set, the deficiencies in setting, placement, gaps, etc. would have been self-evident, just as they were when the station performed a trial placement in 2012.

**Station Actions related to increasing lake levels:**

- Weather Conditions are monitored daily by the Shift Technical Advisor and inputted into Safety Monitor
- Weekend look ahead by Work Week manager for weather effects on weather impact for weekend on Safety Monitor
- Procedurally directed Monthly Recording of Lake Level per PC 80 Part 7 "Lake Level Determination"
- Per PC 80 Part 7 at a Lake Level (580.7 ft.') the Jersey Barriers are installed
- At a lake level of greater than or equal to 588.2 ft the plant will declare an Unusual Event (HU 1.7)
- At a lake level of greater than or equal to 589.2 ft the plant will declare an Alert (HA 1.6)

As described previously, it has been calculated that, with the maximum historical lake rise, it would take greater than 8 weeks to reach the CLB lake level of 581.9 ft. from a starting level of 580.7 ft.

The jersey barriers, including approximately 1000 sand bags, was installed and inspected in less than 24 hours.

**Conclusion**

The time available to respond to rising Lake Michigan pre-storm levels would be at least 8.2 weeks from the time of discovery until the license basis flood level could be attained. During 2012, when the barriers were installed, it took less than 8 hours. Additionally, installation of the modified barrier (which includes approximately 1000 sand bags) was completed in less than 24 hours. Therefore, there is ample time to install the barriers and take appropriate additional actions.