J. Todd Conner
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DTE Energy

10 CFR 50.54(f)

March 8, 2013
NRC-13-0013

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

References:
1) Fermi 2
NRC Docket No. 50-341
NRC License No. NPF-43

2) NRC Letter, "Request For Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident," dated March 12, 2012

3) DTE Electric Letter, "Detroit Edison's 90-Day Response to March 12, 2012 Information Request Regarding Flooding Evaluations and Walkdowns," NRC-12-0036, dated June 8, 2012

Subject: DTE Electric Submittal of Flooding Hazard Reevaluation Report in Response to March 12, 2012 Information Request Regarding Flood Protection Evaluations

On March 12, 2012, the NRC issued Reference 2 to all power reactor licensees and holders of construction permits in active or deferred status. Enclosure 2 to Reference 2 contains specific Requested Actions, Requested Information and Required Responses associated with Recommendation 2.1 regarding flooding evaluations.

In Reference 3, DTE Electric Company* confirmed that Fermi 2 would submit the Flood Hazard Revaluation report by March 12, 2013. The required report is provided in the enclosure to this letter.

The report concludes that the reevaluated flood hazard for Fermi 2 is bounded by the current licensing basis. Therefore, as provided in Reference 2, a flooding integrated assessment for Fermi 2 is not required.

* Previously The Detroit Edison Company

This submittal contains no new regulatory commitments.

Should you have any questions or require additional information, please contact Mr. Kirk R. Snyder, Manager, Industry Interface at (734) 586-5020.

Sincerely,

Enclosure

cc: Director, Office of Nuclear Reactor Regulation
NRC Project Manager
NRC Resident Office
Reactor Projects Chief, Branch 5, Region III
Regional Administrator, Region III
Supervisor, Electric Operators,
Michigan Public Service Commission

I, J. Todd Conner, do hereby affirm that the foregoing statements are based on facts and circumstances which are true and accurate to the best of my knowledge and belief.

J. Todd Conner
Site Vice President, Nuclear Generation

On this 8 day of March, 2013 before me personally appeared J. Todd Conner, being first duly sworn and says that he executed the foregoing as his free act and deed.

Sharon S. Marshall
Notary Public

SHARON S. MARSHALL
NOTARY PUBLIC, STATE OF MI
COUNTY OF MONROE
MY COMMISSION EXPIRES Jun 14, 2013
ACTING IN COUNTY OF Monroe

Enclosure to
NRC-13-0013

Fermi 2 NRC Docket No. 50-341
Operating License No. NPF-43

Fermi 2 Flood Hazard Reevaluation Report

In Response to the 50.54(f) Information Request Regarding Near-Term Task Force Recommendation 2.1: Flooding

FLOOD HAZARD REEVALUATION REPORT

IN RESPONSE TO THE 50.54(f) INFORMATION REQUEST REGARDING NEAR-TERM TASK FORCE RECOMMENDATION 2.1: FLOODING

for the

FERMI 2 NUCLEAR POWER STATION
6400 N. Dixie Highway
Newport, MI 48166
Facility Operating License No. NPF-43
NRC Docket No. 50-341

DTE Energy®

Prepared by:

Black & Veatch

11401 Lamar Ave.

Overland Park, KS 66211

Date: February 27, 2013

	Printed Name/Title	Affiliation	Signature	Date
Preparer:	Steven Thomas / Project Manager	Black & Veatch		2/27/13
Reviewer:	Adam Liebergen / Project Engineer	Black & Veatch		2/27/13
Site Acceptance:	ETHAN HAUSER / PSE	DTE Energy		03/04/13

Contents

1. PURPOSE

1.1 Background

In response to the nuclear fuel damage at the Fukushima-Dai-ichi power plant due to the March 11, 2011 earthquake and subsequent tsunami, the United States Nuclear Regulatory Commission (NRC) established the Near Term Task Force (NTTF) to conduct a systematic review of NRC processes and regulations, and to make recommendations to the Commission for its policy direction. The NTTF reported a set of recommendations that were intended to clarify and strengthen the regulatory framework for protection against natural phenomena.

On March 12, 2012, the NRC issued an information request pursuant to Title 10 of the Code of Federal Regulations, Section 50.54 (f) (10 CFR 50.54(f) or 50.54(f)) (Reference 1) which included six (6) enclosures:

1. [NTTF] Recommendation 2.1: Seismic
2. [NTTF] Recommendation 2.1: Flooding
3. [NTTF] Recommendation 2.3: Seismic
4. [NTTF] Recommendation 2.3: Flooding
5. [NTTF] Recommendation 9.3: EP
6. Licensees and Holders of Construction Permits

In Enclosure 2 of Reference 1, the NRC requested that licensees reevaluate the flooding hazards at their sites against present-day regulatory guidance and methodologies being used for early site permits and combined license reviews'.

On behalf of the DTE Energy Company, this report provides the information requested in the March 12, 50.54(f) letter; specifically, the information listed under the 'Requested Information' section of Enclosure 2, paragraph 1 ('a' through 'e'). As described in Section 4.0, the results from the updated flood evaluations analyses for Fermi 2 are less than the design bases flood protection. Therefore, an interim evaluation is not required and there are no additional actions taken or planned. In addition, based on these results and the guidance in Reference 18, Fermi 2 is not required to perform an Integrated Assessment.

1.2 Requested Actions

Per Enclosure 2 of Reference 1,

> "Addressees are requested to perform a reevaluation of all appropriate external flooding sources, including the effects from local intense precipitation on the site, probable maximum flood (PMF) on stream and rivers, storm surges, seiches, tsunami, and dam failures. It is requested that the reevaluation apply present-day regulatory guidance and methodologies being used for ESP and COL reviews including current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard. The requested information will be gathered in Phase 1 of the NRC staffs two phase process to implement Recommendation 2.1, and will be used to identify potential 'vulnerabilities'." (See definition of vulnerability below.)
>
> "For the sites where the reevaluated flood exceeds the design basis, addressees are requested to submit an interim action plan that documents actions planned or taken to address the reevaluated hazard with the hazard evaluation. "
>
> "Subsequently, addressees should perform an integrated assessment of the plant to identify vulnerabilities and actions to address them. The scope of the integrated assessment report will include full power operations and other plant configurations that could be susceptible due to the status of the flood protection features. The scope also includes those features of the ultimate heat sinks (UHS) that could be adversely affected by the flood conditions and lead to degradation of the flood protection (the loss of UHS from non-flood associated causes are not included). It is also requested that the integrated assessment address the entire duration of the flood conditions."

A definition of vulnerability in the context of [enclosure 2 of Reference 1] is as follows: "Plant-specific vulnerabilities are those features important to safety that when subject to an increased demand due to the newly calculated hazard evaluation have not been shown to be capable of performing their intended functions."

1.3 Requested Information

Per Enclosure 2 of Reference 1, the final report should be provided documenting results, as well as pertinent site information and detailed analysis, and include the following:

a. "Site information related to the flood hazard. Relevant SSCs important to safety and the UHS are included in the scope of this reevaluation, and pertinent data concerning these SSCs should be included. Other relevant site data includes the following:

    i. Detailed site information (both designed and as-built), including present-day site layout, elevation of pertinent SSCs important to safety, site topography, as well as pertinent spatial and temporal data sets;

    ii. Current design basis flood elevations for all flood causing mechanisms;

    iii. Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance;

    iv. Changes to the watershed and local area since license issuance;

    v. Current licensing basis flood protection and pertinent flood mitigation features at the site;

    vi. Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results, etc.)"

b. "Evaluation of the flood hazard for each flood causing mechanism, based on present-day methodologies and regulatory guidance. Provide an analysis of each flood causing mechanism that may impact the site including local intense precipitation and site drainage, flooding in streams and rivers, dam breaches and failures, storm surge and seiche, tsunami, channel migration or diversion, and combined effects. Mechanisms that are not applicable at the site may be screened-out; however, a justification should be provided. Provide a basis for inputs and assumptions, methodologies and models used including input and output files, and other pertinent data."

c. "Comparison of current and reevaluated flood causing mechanisms at the site. Provide an assessment of the current design basis flood elevation to the reevaluated flood elevation for each flood causing mechanism. Include how the findings from Enclosure 2 of the 50.54(f) letter (i.e., Recommendation 2.1 flood hazard reevaluations) support this determination. If the current design basis flood bounds the reevaluated hazard for all flood causing mechanisms, include how this finding was determined."

d. "Interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment described below, if necessary."

e. "Additional actions beyond Requested Information item 1.d taken or planned to address flooding hazards, if any."

2. SITE INFORMATION

This section of the FHRR provides a description of the site location and description, a summary of current design basis flood elevations, flood protection, and changes to the site and vicinity subsequent to the design basis flood analysis that have the potential to impact the predicted flood elevations. Description of the current design basis flood elevations and flood protection is based on information in the Fermi 2 Updated Final Safety Analysis Report (UFSAR) (Reference 2); specific sections are identified below.

2.1 Site Location and Description

The Fermi site is located adjacent to the western shore of Lake Erie. Prior to construction of Fermi 2, the site area was a lagoon separated from Lake Erie by a barrier beach, known as Lagoona Beach, which formed the eastern site boundary. The lagoon was connected to Lake Erie by Swan Creek, a perennial stream that discharges into Lake Erie about 1 mile north of the Fermi plant site. The site for Fermi 2 was prepared by excavating soft soils and rock, and constructing rock fill to a nominal plant grade elevation of 583 ft. All elevations refer to New York Mean Tide, 1935. The topography of the developed site as of December 10, 1972, is shown in Reference 2, Figure 2.4-3. [Reference 2, Section 2.4.1.1, 1st Paragraph]

"Category I structures housing safety-related equipment consist of the reactor/auxiliary building and the residual heat removal (RHR) complex." As described in Reference 2, Section 9.2.5, the ultimate heat sink is provided by the RHR complex. "The plant site is not susceptible to flooding caused by surface runoff because of the shoreline location and the distance of the site from major streams. Plant grade is raised approximately 11 ft above the surrounding area to further minimize the possibility of flooding. Flooding of the site is conceivable only as the result of an extremely severe storm with a storm-generated rise in the level of Lake Erie. Protection of safety-related structures and equipment against this type of flooding is provided through the location, arrangement, and design of the structures with respect to the shoreline and possible storm-generated waves." [Reference 2, Section 2.4.1.1, 2nd Paragraph]

"Roof water that is collected through drainage systems from all structures and catch basins inside the perimeter road is collected and routed to the station storm-water drain system to prevent ponding of water adjacent to structures. Water in the plant storm-water drain system is then discharged into the overflow canal. In grassy areas outside the perimeter road, and in gravel areas, catch basins discharge water into the quarry run fill. In paved areas, the catch basins are usually tied to the storm-water drain system." [Reference 2, Section 2.4.1.1, 4th Paragraph]

The Fermi site is in the Swan Creek drainage basin. As described in the Reference 2, the watershed is an area of approximately 109 sq. mi., elongated in shape from northwest to southeast. It is noted that the updated analyses of flooding in the Swan Creek drainage basin performed for Fermi 3 (Fermi 3 FSAR - Reference 3) used a drainage basin area of 106 sq. mi. The associated NRC Safety Evaluation Report (SER) for Chapter 2 of the Fermi 3 FSAR (Reference 4) used a drainage basin area of 101 sq. mi., and concluded that the Fermi 3 FSAR is conservative. The basin is about 25 miles long with a maximum topographic relief of about 130 ft. The drainage area topography is flat to gently undulating and varies from about 700 ft elevation in the upper watershed to about 570 ft elevation at Lake Erie. [Reference 2, Section 2.4.1.2.2]

2.2 Design Basis Flood Elevations

Three different flood scenarios are evaluated as part of Fermi 2 licensing basis [Reference 2, Section 2.4.2.2.1]:

- Local Intense Precipitation
- Flooding in Streams and Rivers
- Storm Surge

A summary of the results as described in Reference 2 for each of these flooding scenarios is provided below; specific sections from Reference 2 are identified with the associated discussion.

2.2.1 Local Intense Precipitation

Local probable maximum precipitation (PMP) runoff on the plant site coincident with runoff from the 2-square mile area above the plant site, assuming blockage of plant drainage, would not result in adverse effects on the safety-related (Category I) facilities. This conclusion is based on an estimated Probable Maximum Flood (PMF) of 25,300 cfs on the adjacent area resulting in a water elevation of less than 582 ft, and the 15-minute PMP of 4.9 in. over the plant site. Temporary local water buildup on the plant site due to the failure of the plant drainage system will flow into the lower land and swamps at the northern end of the plant area and eventually discharge into Lake Erie through estuaries. The local temporary water buildup elevation will be substantially lower than the flood elevation attributed to the maximum storm surge. [Reference 2, Section 2.4.2.2.1.c]

2.2.2 Flooding in Streams and Rivers

A PMF scenario on Swan Creek is considered. The PMF is an estimated flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMF on Swan Creek was estimated as the maximum flood runoff resulting from a PMP occurring on the entire drainage basin of 109 sq. mi. [Reference 2, Section 2.4.3]. It is noted that the Fermi 2 PMF was based on a Swan Creek drainage area basin of 109 sq. mi. Subsequently, analyses in support of the Fermi 3 FSAR determined that the drainage area basin is approximately 106 sq. mi. [Reference 3, Section 2.4.1.2.1]. The PMF of 89,000 cfs on Swan Creek coincides with the mean monthly maximum water level of 575.3 ft in Lake Erie. The resulting PMF flow elevation of 577.3 ft would provide a safety margin of 5.7 ft. Even by the use of a conservative slope/area computation, the PMF elevation would be less than 582 ft. [Reference 2, Section 2.4.2.2.1.a]

2.2.3 Storm Surge

The potential for flooding due to a wind driven storm surge (Probable Maximum Meteorological Event – PMME) from Lake Erie was determined. The maximum PMME wind tide of 11.4 ft was calculated for the Fermi site with the PMME wind speeds. As an additional conservatism, a wind tide of 11.6 ft was used for design purposes. A total stillwater elevation of +16.4 ft was selected as the design maximum. This was based on the PMME with a storm path along the axis of Lake Erie (N67.5°E). Elevation +16.4 ft results from a calculated wind tide of +11.6 ft superimposed on a maximum monthly mean lake level of +4.8 ft (above the Low Water Datum of 570.5 ft). This storm surge would occur at the Fermi site approximately 9 hr after the maximum wind reaches the shore. The resulting stillwater flood elevation at the plant site area in this case is 586.9 ft, or 3.9 ft above the plant grade elevation. [Reference 2 Section 2.4.5.3]

During the occurrence of the PMME, incident waves attacking the shoreline can be transmitted inland across the flooded plant grade. The wave heights depend on the available water depth above plant grade, the incident wave characteristics attacking the shoreline, the configuration of the shore barrier, and the location and configuration of other obstacles. [Reference 2, Section 2.4.5.4.2.3, 1st Paragraph] A rock shore barrier is constructed along the shore in front of Fermi 2. The rock shore barrier crest elevation is 583 ft nominal and the toe elevation is 572 ft nominal. [Reference 2, Section 2.4.5.4.2.3, 2nd Paragraph] The shore protection is not credited to preclude flooding of these structures. The maximum supported wave height is the controlling factor for plant structures located more than a few hundred feet inland from the shoreline. [Reference 2 Section 2.4.5.4.2.3, 4th Paragraph]

"Waves that are transmitted over the shore barrier will attack the office service and radwaste buildings of Fermi 2. These buildings are not Category I structures and, therefore, could be damaged during the storm without causing a safety concern to the public." [Reference 2, Section 2.4.5.4.2.3, 5th Paragraph]

"Small waves can reach the Category I structures by traveling around the northerly and southerly ends of the shore barrier. Waves traveling around the ends of the shore barrier undergo several effects, including the following:

a. Breaking caused by the shallow depths of the flooded plant grade.
b. Diffraction around the ends of the other plant structures.
c. Reflection off plant structures before reaching the Category I structures.
d. Reduction caused by plant grade bottom friction and side friction of obstructing structures." [Reference 2, Section 2.4.5.4.2.3, 6th Paragraph]

"Waves approaching the north end of the shore barrier will be reduced to the maximum inland support wave heights of 3.0 and 5.4 ft for plant grade Elevations 583.0 and 580.0 ft, respectively, in approaching Category I structures. Waves approaching the southerly end of the shore barrier will be reduced in height approaching Category I structures as a result of the maximum inland supported wave height and the protection provided by the office service and turbine buildings. Neglecting any reduction effects from protection provided by the office service and turbine buildings, waves approaching Category I structures from the south will be reduced to the maximum inland supported wave height of 3.0 ft for the plant grade elevation of 583.0 ft." [Reference 2, Section 2.4.5.4.2.3, 7th Paragraph]

"Maximum runup elevations on the exposed north faces of the reactor/auxiliary building and the RHR complex are 593.0 and 598.0 ft for the 3.0-ft and 5.4-ft waves, respectively. The maximum runup elevation on the exposed south faces of the reactor/ auxiliary building and the RHR complex, the exposed east face of the RHR complex, and the west face of the reactor/auxiliary building is 593.0 ft for the 3.0-ft wave. This wave could possibly reach the west face of the reactor/auxiliary building by reflection from the east face of the RHR complex. The east face of the reactor/auxiliary building is not exposed to waves and wave runup. The west face of the RHR complex is landward of the storm direction and not subject to waves and wave runup. As previously stated, no shore protection is required to preclude flooding of these structures." [Reference 2, Section 2.4.5.6.2]

2.2.4 Summary

The water level at the site is controlled by Lake Erie. The PMF flow from Swan Creek has no significant effect on the design water level at the site. The design basis flood water level is established at the maximum lake stillwater level due to storm surge of Elevation 586.9 ft, or 3.9 feet above plant grade elevation. As described in Reference 2, Section 2.4.2.2.1.e, "At plant grade elevation (583 ft.), the lake water would extend approximately 2.5 miles inland from the plant site (Reference 2, Figure 2.4-11) and even further inland at maximum stillwater level." The impacts from waves and wave run-up are considered as part of the design basis.

2.3 Flood Protection

"The Category I reactor/auxiliary building, which houses safety-related systems and components, is designed against flooding to Elevation 588.0 ft, or 1.1 ft above the PMME stillwater flood elevation of 586.9 ft. All doors and penetrations through the outside walls below the design flood elevation are of watertight design. All safety-related systems and equipment located inside this Category I structure are protected from the PMME flood. The reactor/auxiliary building is designed to withstand wave action associated with this flooding. All interior floor drain systems inside the reactor/auxiliary building are not connected to the yard storm drainage system and, therefore, no potential water backflow into the structure is anticipated during the design flood condition. Shore protection is not required to preclude flooding of this structure. The reactor/auxiliary building has only a few essential penetrations in the exterior walls. Penetrations below Elevation 588 ft are watertight. The presence of the turbine building prevents waves and wave runup above the sill elevations on the east wall of the reactor/ auxiliary building, thereby preventing flooding of the buildings. The south wall of the reactor/auxiliary building has two large openings and several waterproofed pipe-sleeved openings. These large openings are in an air-locked rail-car door and an air-locked personnel door. Both of these doors are air-locked and completely waterproofed to preclude wave runup flooding." [Reference 2, Section 2.4.2.2.2]

"The RHR complex is watertight to Elevation 590.0 ft. The north, south, and west walls have no openings. The east wall has approximately 30 waterproofed pipe-sleeved openings. The east wall also has four sets of double 3 ft by 7 ft doors for access to the building. These doors are normally closed and locked, and have their thresholds at Elevation 590.0 ft and extend to Elevation 597.0 ft. They are of steel construction and are shielded behind reinforced-concrete missile walls. The east wall also has eight 4" diameter openings with water tight seals located within each of the two RHR cable vaults at elevations above 590'-6". Waves reaching the east wall of the RHR complex across the flooded site would be diminished considerably by the stairs, the missile wall, and the landing at Elevation 590.0 ft in front of the doors. The insignificant amount of runup above the flooded elevation of 586.9 ft, or generated by the reduced waves, may find its way through the door threshold and door jambs, at Elevation 590.0 ft, and be diverted into the floor drain system in the building. The structure is also designed to withstand the wave action associated with this flooding. Shore protection is not required to preclude flooding of this structure." [Reference 2, Section 2.4.2.2.3]

"The Category I piping and electrical ducts between the RHR complex and the reactor building are buried; and thus, below the site flood elevation of 586.9 ft during the PMME. The RHR supply, RHR return, and emergency equipment service water pipelines to both divisions will continue to function during the flood." "The minimum elevation for cable termination in either the RHR complex or reactor building is 588.7 ft, which is above the site maximum probable stillwater elevation of 586.9 ft." [Reference 2, Section 2.4.2.2.4]

In response to NRC request (Reference 1) DTE Energy performed walkdowns of Fermi 2 flood protection features. These walkdowns and results are summarized in Reference 17; which includes the following conclusions:

- "No differences or contradictions in flood hazard levels were found in design or licensing basis documentation." [Reference 17, Enclosure 1, Page 5]

- "The results of the walkdown, which are further discussed in section (f.), indicate that the overall effectiveness of the Fermi 2 flood protection features to perform their credited CLB function, described in sections (a.) and (b.), to be adequate. The aggregate effect of the "deficiencies" to the plant external flood protection procedures, features, and their associated actions identified in this report would not prevent their design functions from being performed as credited in the Current Licensing Basis (CLB). No additional existing plant equipment, structures, or procedures, not part of the flooding CLB are considered for use to mitigate an external flood." [Reference 17, Enclosure 1, Page 10]

2.4 Flood Related Changes to the License Basis

Changes to flood protection features since the initial issuance of the operating license are the following:

- The addition of redundant check valves and manual isolation valves to prevent backflow flooding into the reactor building. [Reference 2, Section 3.4.4.4.2]

- Redesign of the Category I ductbanks between the RHR complex and the Reactor/Auxiliary building. [Reference 2, Section 3.4.4.3]

As part of this FHRR a review was performed using site drawings and aerial photographs to identify changes that have been made at Fermi 2 subsequent to the evaluations described in Reference 2 that could impact the flood hazard evaluation. This review identified that changes made to the security system at Fermi 2 could impact the local intense precipitation and wave run-up evaluations. Specifically, subsequent to the design basis flood analyses for Fermi 2, concrete barriers were erected in the vicinity of the North and South Protected Area fence-lines.

Fermi 2 is co-located with a COL plant (Fermi 3). Figure 1 shows the proposed location for Fermi 3 relative to Fermi 2. Due to the proximity of Fermi 3 to Fermi 2, specific flooding analyses performed for Fermi 3 can be used to determine the updated flood hazard to Fermi 2. Specifically, the analyses for Fermi 3 for the PMF on Swan Creek and determination of the maximum stillwater elevation during the wind driven surge from Lake Erie can be directly applied to Fermi 2. Thus, in lieu of performing a review to identify changes to the local area watershed, it is recognized that changes that would potentially affect the results of the design bases analyses for the PMF on Swan Creek or the wind driven surge for Fermi 2 would be captured as part of the updated analyses performed for Fermi 3.

There are two flooding potential scenarios for Fermi 2 that are not addressed by the associated Fermi 3 analyses:

- Local Intense Precipitation
- Wave Run-Up During the Wind Driven Surge from Lake Erie

Analyses for these two events were performed for Fermi 3; however, due to differences in grade elevation for Fermi 3 vs. Fermi 2, the Fermi 3 analyses for these two events cannot be applied to Fermi 2. Evaluations were performed for these two events as part of the Fermi 2 licensing bases, however, the results from these two events could be affected by the changes to the security barriers due to the following.

- For the local intense precipitation event, the barriers can restrict surface drainage from leaving the Fermi 2 plant vicinity in the North and South directions, but would not impact flow towards the East or West. The updated analysis for the local intense precipitation event, described below, includes consideration of these concrete barriers.

- In the evaluation of waves and wave run-up during a wind driven surge event, the barriers can impact the heights of the waves. The updated analysis for the wave run-up event, described below, includes consideration of the concrete barriers.

Analyses for these two events are described in Section 3, below.

3. SUMMARY OF FLOOD HAZARD REEVALUATION

3.1 Local Intense Precipitation

As described in Section 2.2.1, above, the Fermi 2 licensing basis evaluates the potential impact from a local precipitation flood event. This includes evaluations of runoff from the area to the west of the plant site and directly on the plant site. Analyses of the local precipitation event was also performed for the Fermi 3 COLA, but, due to elevation differences between Fermi 3 and Fermi 2, cannot be applied to Fermi 2. Thus, a new local precipitation flood analyses was performed for Fermi 2 [Calculation 177910.51.1001]. The following discussion in Section 3.1 is from Calculation 177910.51.1001 (Reference 19).

The analysis determines the maximum water surface elevation reached as a result of the Local Intense PMP for the Fermi 2 site. The site and adjacent areas were analyzed separately. One analysis considers a 1.78 sq. mi. area to the west of the site that drains towards and around the Fermi 2 site. The 1.78 sq. mi. area was determined based on topographical maps. The second analysis considers the actual 51 acre on-site drainage area. These two areas are shown on Figure 2. Local intense Probable Maximum Precipitation rates (PMP) were calculated, peak runoff values were calculated, and maximum water surface elevations were determined for both areas.

The hydrological analysis used for estimating the Probable Maximum Flood (PMF) for the local contributing watershed area (1.78 sq. mi.) and the Fermi 2 site (51 acres) was performed using information from Hydrometeorological Reports HMR 51 and 52 (References 7 & 8) to determine the PMP. The contribution from the equivalent maximum snowmelt intensity was added to the PMP to maximize the peak runoff. The Rational Method was used to estimate the resulting PMF peak runoff from both sites. The HEC-RAS 4.0 software was used to determine resulting flood elevations for the 1.78 sq. mi. watershed and Manning's equation was used to compute the maximum water surface elevation on the 51 acre site area. The evaluation was conducted to provide conservative results of the PMP and resulting PMF elevations. Discussion of the analysis for the local contributing watershed area (1.78 sq. mi.) and the Fermi 2 site (51 acres) is provided below.

To develop a conservative local intense PMP, the methodology for determining the PMP for a 1 hr 1 sq. mi. storm (Reference 8) was used. The PMP Duration-Intensity Curve shown on Figure 3 was derived through the multiplication of the intensity determined for the 1 hr 1 sq. mi. PMP and the percentage multipliers for the 5, 15, and 30 minute increments as shown in Reference 8, Figures 36 to 38.

The Time of Concentration (T_c) and the associated PMP intensity for both the 1.78 sq. mi. local watershed and the 51-acre plant site are then determined using the methodology in Reference 11 was used to determine T_c. The T_c was then used to select the appropriate PMP intensity from the Duration-Intensity Curve (Figure 3).

Reference 11 provides methods for determining T_c, which is defined as the time for runoff to travel from the hydraulically most distant point within the watershed to a point of interest within the watershed. T_c is a function of land cover, flow path length and flow path slope and is determined for various types of flow depending on the topography and length of flow path. The total time of concentration is determined using the following equation.

Total Time of Concentration

$$T_c = T_{c1} + T_{c2} + T_{c3}$$

Where: T_{c1}= Sheet flow time of concentration, (minutes)
T_{c2}= Shallow concentrated flow time of concentration, (minutes)
T_{c3}= Open channel flow time of concentration, (minutes)

The methods in Reference 11 to determine time of concentration for sheet flow, shallow concentrated flow, and open channel flow are shown below.

Sheet Flow (Reference 11, Equation 3-3)

$$T_{c1} = \frac{[0.007(L \cdot n)^{0.8}]}{P_2^{0.5} \cdot s^{0.4}}$$

Where: T_{c1}= Time of concentration, (hours)
n = Manning roughness coefficient (Reference 11, Table 3-1, cultivated soils: residue cover ≤ 20%)
L = Flow length, ft
P_2= 2-year, 24-hour rainfall, inches (Reference 10, Isopluvial Chart 44)
s = Overland slope, ft/ft

Shallow Concentrated Flow (Reference 11, Equation 3-1)

$$T_{c2} = \frac{L}{V}$$

Where: T_{c2}= Shallow concentrated flow time of concentration, (seconds)
L= Flow length, ft
V= Average velocity, ft/s (Reference 11, Figure 3-1)

<u>Open Channel Flow (Reference 11, Equations 3-4 and 3-1)</u>

$$V=\frac{1.49r^{\frac{2}{3}}s^{\frac{1}{2}}}{n}$$

$$T_{c3}=\frac{L}{V}$$

Where: V= Average velocity, ft/s
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow (Reference 12)

The sum of the sheet flow and shallow concentrated flow components is limited to a maximum of 15 min per Reference 13. The open channel flow calculation was used to determine a T_c of 25.64 min from the start of the streambed to the edge of the wetland area which is considered the start of the channel that will to convey flood flow past the plant site. Thus, the total time of concentration is 40.64 minutes (15 + 25.64 mins). To ensure a conservative flood evaluation a total time of concentration of 15-mins is used, resulting in a local Intense PMP of 36.7 in/hr for the 1.78 sq. mi. watershed, as shown on Figure 3.

To evaluate a limiting case scenario the additional contribution that would result should the local intense PMP event occur simultaneous with the presence of significant snowpack in the local watershed is considered. The scenario considers snow accumulation through the winter. At the beginning of spring a saturated air mass with high temperature produces a rainfall event of the magnitude of the PMP. Inspection of historical monthly temperature averages and maxima showed that April is a month where relatively high temperatures can occur after freezing or lower temperatures have been maintained. These conditions are required for the PMP on snow to occur, therefore the event was assumed to occur in April.

The analysis was conducted following the procedures outlined in the U.S. Army Corps of Engineers EM 110-2-1406 "Runoff from Snowmelt" (Reference 9), as indicated in ANSI/ANS 2.8-1992 (Reference 5). The lumped model approach was used, which assumes the progression of each variable through time can be a single computational algorithm that represents the entire basin. The sources of energy affecting snowmelt include short-wave and long-wave net radiation, convection from the air, vapor condensation, conduction from the ground, and energy contained in rainfall.

The critical sequence of meteorological factors affecting snowmelt is described in the explanation of the equation used. Whenever sufficient energy is available, some snow (ice) will melt and form liquid water (snowmelt). Snowmelt is held as a liquid in the interstices between snow grains and will increase snow water content. Snowpack is considered "ripe" when it is isothermal at 0°C (32°F) and saturated. Estimates of snowmelt amounts are derived through the use of energy balance equations. Based on the site location and vicinity, the equation for open or partly forested areas is used (Reference 9, Equation 5-19):

$$M = (0.029+0.0084kV+0.007Pr)(Ta-32)+0.09$$

Where, M = snowmelt, in/day
k = basin wind coefficient
V = wind velocity, miles/hr
Pr = rate of precipitation, in/day
Ta = temperature of saturated air, °F

This equation assumes constant shortwave radiation and snowmelt. A value of one was used for the basin wind coefficient k, which represents unforested plains. Precipitation rate (Pr) was the local intense PMP described above. For the purposes of this analysis, the initial snowpack is assumed to cover the entire watershed with no significant variation of temperature or snow depth. It is conservatively assumed that there is an unlimited amount of snow available to melt.

The wind velocity and temperature were calculated by performing a statistical analysis of historical wind velocities and dewpoints measured in April at the Detroit Metropolitan Airport meteorological station. During a PMP event it is assumed that rainwater temperature is equal to air temperature and that dewpoints are representative of air temperature. Maximum hourly windspeeds and dewpoints for the month of April were compiled for each year on record (34 years, 1961-1995). An extreme value frequency analysis was conducted to determine the hourly windspeed with an annual (in April) exceedance probability of 50% (2-yr hourly windspeed) and the hourly dewpoint with an annual (in April) exceedance probability of 1% (100-yr dewpoint). The estimated max 2-yr windspeed and the 100-yr dewpoint are respectively 52.3 km/h (32.5 mph) and 20.6°C (69.1°F).

Using these windspeed and temperature values, a k of 1, and the local intense PMP values for the 1.78 sq. mi. local area watershed and 51 acre plant site, the snowmelt was determined to be 9.99 in/hr for the local area watershed and 18.8 in/hr for the plant site.

The limiting scenario is for the PMF that results when the PMP occurs when there is significant snowpack. The Rational Equation was used to determine the PMF flow (Q) for both the 1.78 sq. mi. local watershed and the 51 acre plant site under such a scenario.

$$Q = CIA$$

Where: Q = flow rate for the PMF (ft^3/sec)
C = runoff coefficient constant = 1.0, conservatively assumes 100% runoff
I = intensity in/hr (the local intense PMP + snowmelt (M))
A = drainage area in acres

The results are summarized in Table 3.1-1. below.

Table 3.1-1
PMF Flow Rates

Area	PMP Intensity (in/hr)	Snowmelt (M) (in/hr)	Flow Rate (Q) (cfs)
1.78 sq. mi. local area watershed	36.7	9.99	53,277
51 acre plant site	70.6	18.8	4,559

The hydraulic evaluation for the 1.78 sq. mi. local area watershed and the 51 acre site are described below.

3.1.1 Local Contributing Watershed Hydraulic Evaluation

Review of the site and vicinity topographical maps shows that the area to the west of the site drains toward the site then could divide (based on water elevation) as it travels to the north (referred to as the North Reach) and south (referred to as the South Reach) around the site to Lake Erie. The cross sections for the North and South Reaches were determined based on site topographical maps and supplementing information from the ArcGIS Map Service US Topo. The North and South Reaches share a common cross section (referred to as River Station 100 in the modeling). A conservative approach of truncating the cross section before it crosses the site was used. This conservatively reduced the available conveyance area, thus producing a maximum water surface elevation. The HEC-RAS steady state model was used to determine a maximum water surface elevation that will be assumed to reach across the site.

Both the North and South Reaches eventually flow into Lake Erie; i.e., the downstream boundary condition for this analysis. The 100-year level for Lake Erie (576.3 ft) (Reference 3) is used as the downstream boundary condition for the hydraulic analysis of the 1.78 sq. mi. area.

The balance of flow between the North and South Reach was determined using an iterative approach until a common water surface elevation occurs at the most upstream cross section for both reaches and the total of the two flows equals the total peak flow discharging from the 1.78 sq. mi. site. The most upstream cross section (River Station 100) produces the highest water surface elevation. This cross section would be used to report the maximum water surface elevation that the site may encounter for a local intense PMP on the contributing 1.78 sq. mi. site. The calculated highest water surface elevation is 582.8 ft and is considered the maximum water surface elevation produced from the runoff of the 1.78 sq. mi. contributing area. This maximum water surface elevation is less than the height of the plant grade of 583 ft.

3.1.2 Fermi 2 Site Hydraulic Evaluation

Figure 4 shows the Fermi 2 site area with the buildings as a backdrop for reference. For the hydraulic evaluation of the Fermi 2 site, Figure 5 shows the first cross section located at the western edge of the site. This location is capturing the runoff from the majority of the site and passing it through the cross section. The channel bottom elevation used for the cross section in this area is 582 ft. The subarea contributing to the cross section is the area that is at elevation 582 ft or higher east of the cross section. The area below 582 ft is considered to drain off the site without passing through the cross section. Figures 4 and 5 also show the obstructions, the watershed boundary, the contours, and the location for the slope calculation.

Inputs to this calculation are as follows:

- For this location the majority of the site area drains to the cross section, measured to be 43 acres.
- The site is an industrial site with asphalt paving or gravel areas. A Manning's "n" value of 0.02 is used to represent the channel. This would correspond to a gravel bottom or rough concrete per Ven Te Chow, Open Channel Hydraulics, 1959, (Reference 14).
- A representative slope going from approximately the middle of the site to western edge of the site was used. Based on a 1.5 ft drop across a 797 ft length, this slope is 0.0019 ft/ft.
- Accounting for obstructions, the unobstructed length of this cross section is 692 ft.
- A Runoff Coefficient of 1.0 is used to represent full pavement coverage.
- All of the site drainage is assumed to be plugged.

The flow depth was calculated using Manning's Equation for flow rate.

$$Q = (1.486/n)(A)(R)^{2/3}(S)^{1/2}$$

Where: Q = flow rate (cfs)
n = Manning coefficient of roughness
A = cross sectional area of flow (sqft)
R = hydraulic radius (ft)
S = slope of channel (ft/ft)

A flow depth of 1.4 ft is calculated, for a maximum water surface elevation of 583.4 ft; which is less than the height of the door sills for the Class I structures (583.5 ft).

3.2 Flooding in Streams and Rivers

As described in Section 2.2.2, above, the Fermi 2 licensing basis evaluates the potential impact from flooding from Swan Creek. Swan Creek is the outlet for the Swan Creek watershed and a minor tributary of the western basin of Lake Erie. As shown on Figure 1, Swan Creek is located along the northern side of the Fermi site boundary.

The potential for flooding in Swan Creek was also evaluated as part of the Fermi 3 COLA, specifically, Reference 3, Section 2.4.3. As shown on Figure 1, both Fermi 3 and Fermi 2 are located relatively close together and would be impacted equally by flooding in Swan Creek. Due to the relative proximity of both units to each other and to Swan Creek, and the analysis methods used to determine the probable maximum flood (PMF) for Fermi 3, the PMF for Fermi 3 can also be applied to Fermi 2.

The PMF analyses for the Fermi 3 COLA are described in Reference 3, Section 2.4.3. The analysis for the in Reference 3 included conservative inputs and assumptions for precipitation (including snowmelt), runoff modeling, and analysis methods. Details are described in Reference 3, Section 2.4.3. The NRC review of the Swan Creek PMF is documented in Reference 4, Section 2.4.3.

Consistent with ANSI/ANS-2.8-1992 (Reference 5), Section 9.2.3.2, the PMF analysis in Reference 3 considers three different alternative event combinations (i.e., combined effects flooding), shown below.

- Alternative I –
  - One-half PMF or 500-year flood, whichever is less, plus
  - Surge and seiche from worst regional hurricane or windstorm with wind wave activity, plus
  - 100-year or maximum controlled level of waterbody, whichever is less.

  The calculated flood level for Alternative I is 580.6 feet, 2.4 feet below plant grade.

- Alternative II –
  - PMF, plus
  - 25-year Surge and seiche with wind wave activity, plus
  - 100-year or maximum controlled level of waterbody, whichever is less.

  The calculated flood level for Alternative II is 579.5 feet, 3.5 feet below plant grade.

- Alternative III –
  - 25-year flood, plus
  - Probable maximum surge and seiche with wind wave activity, plus
  - 100-year or maximum controlled level of waterbody, whichever is less.

The calculated flood level for Alternative III is 586.6 ft; which is 0.3 ft below the current design bases flood level of 586.9 ft, 1.4 ft and 3.4 ft below the flood stillwater elevation that the Category I Reactor/Auxiliary building and RHR complex, respectively, are designed for. The much higher flood elevation for Alternative III is based on including the probable maximum surge. As shown in Section 3.4, below, the calculated Stillwater elevation for the probable maximum surge is 586.6 feet. During this event, as described in Section 2.2.4, the stillwater elevation is located inland to the west of the site, such that the flow in Swan Creek has negligible impact on the water elevation at the plant site.

As shown on Figure 1, the projected location for Fermi 3 is relatively close to Fermi 2; which are both located in similar proximity to Swan Creek. The distance from Swan Creek to Fermi 3 is not an input to the PMF analysis. Thus, distance differences between Swan Creek to Fermi 3 vs. Swan Creek to Fermi 2 do not impact the analysis results. Therefore, the PMF analysis for Fermi 3 is applicable to Fermi 2. As discussed above, the highest predicted PMF flood elevation (combined effects flood) is 586.6 feet and is less than the current licensing basis maximum flood elevation for Fermi 2.

3.3 Dam Breaches and Failures

Based on information available from the Corps of Engineers National Inventory of Dams (NID) website, https://nid.usace.army.mil (Reference 23), there are no dams in the Swan Creek watershed.

Furthermore, regarding dam breaches or failures, Reference 3, Section 2.4.4, states:

> "The water supply for Fermi 3 is from Lake Erie. The outflow from Lake Erie is not regulated. The outflow from Lake Erie is controlled exclusively by the hydraulic characteristics of the outlet rivers. Thus, there are no dam failures that could impact the water supply for Fermi 3.
>
> "Fermi 3 is located within the Swan Creek watershed. The Swan Creek watershed contains no dams upstream or downstream within the vicinity of Fermi 3. Thus, there are no dam failures that could result in flooding to the Fermi 3 site. Additionally, there are no water control structures erected on the Fermi site whose failure would cause potential flooding.
>
> "Therefore, there are no potential dam failures that could affect Fermi 3 safety-related structures or components."

Regarding flooding due to dam failures, the NRC Safety Evaluation Report for Fermi 3 FSAR Chapter 2 (Reference 4) Section 2.4.4.4, states:

> "The staff reviewed FSAR Section 2.4.4, Potential Dam Failures. In Section 2.4.3.4 of the FSAR, the second paragraph states that "There are no dams existing within the Swan Creek watershed ..." In response to this statement, the NRC staff request the applicant to provide additional information on the justification for the statement regarding dams in the watershed through RAI 2.4.4-1 (ML092790561, dated September 30, 2009). The RAI specified that the applicant should demonstrate that a reasonable search of records or applicable databases has been conducted

to support its conclusion. In response to RAI 2.4.4-1, the applicant referenced the USACE National Inventory of Dams database. The staff checked the National Inventory of Dams on October 21, 2009 and verified that there are no dams within the Swan Creek Watershed (USACE, 2007). The staff verified that the information in the dam inventory and finds that there is no risk of flooding due to a potential dam failure. For the reasons given above, the staff concluded that the identification and consideration of the effects of dam failures at the site and in the surrounding area are acceptable and meet the requirements of 10 CFR 52.79, 100.23(d), and 100.20(c)."

As both Fermi 3 and Fermi 2 are on the same site and share the same major hydrologic features (i.e., Lake Erie and the Swan Creek Drainage Basin), this same rationale is applicable to Fermi 2. Therefore, there are no potential dam failures that could affect Fermi 2 safety-related structures or components.

3.4 Storm Surge, including Wind-Wave Activity

As described in Section 2.2.3, above, the Fermi 2 licensing basis evaluates the potential impact from a wind driven surge from Lake Erie. Lake Erie is the dominant hydrological feature at the site. As shown on Figure 1, Lake Erie is located along the eastern side of the Fermi site boundary.

A wind driven surge event from Lake Erie is also evaluated as part of the Fermi 3 COLA, specifically, Reference 3, Section 2.4.5. As shown on Figure 1, both Fermi 3 and Fermi 2 are located relatively close together and would be impacted equally by a wind driven surge from Lake Erie. Due to the relative proximity of both units to each other and to Lake Erie, and the analysis methods used to determine the probable maximum surge for Fermi 3, the probable maximum surge for Fermi 3 can also be applied to Fermi 2.

The probable maximum surge and seiche flooding analyses for the Fermi 3 COLA are described in Reference 3, Section 2.4.5. The analysis was performed using the STWAVE and ACES computer models. The analysis for the Fermi 3 COLA included conservative inputs and assumptions.

Consistent with ANSI/ANS-2.8-1992 (Reference 5), Section 9.2.3.1, the following combination of flood causing events were used to provide an adequate design bases for a shore location such as the Fermi site.

- Probable maximum surge and seiche with wind-wave activity.
- 100-year or maximum controlled level of waterbody, whichever is less.

The calculated flood level for this combination of flood causing events is 586.6 ft; which is 0.3 ft below the current design bases flood level of 586.9 ft. This water elevation is 1.4 ft and 3.4 ft below the flood stillwater elevation that the Category I reactor/auxiliary building and RHR complex, respectively, are designed for.

As shown on Figure 1, the projected location for Fermi 3 is relatively close to Fermi 2; which are both located in similar proximity to Lake Erie. Due to the topography of the site and the nearby vicinity, during the probable maximum surge event, the stillwater elevation would extend well inland to the west of the site (see discussion in Section 2.2.4). Thus, the stillwater elevation will be the same at Fermi 2 as the analytical results for Fermi 3. Therefore, the probable maximum wind-driven surge flood analysis for Fermi 3 is applicable to Fermi 2. As discussed above, the highest predicted flood elevation due to storm surge is 586.6 ft and is less than the current licensing basis maximum flood elevation for Fermi 2.

Due to design requirements imposed by the selected Design Control Document (GEH ESBWR) for Fermi 3, there are differences between the plant grade elevation for Fermi 3 vs. Fermi 2. Due to these differences in plant grade elevation the analysis of wave activity for Fermi 3 cannot be directly applied to Fermi 2. Therefore, a new analysis of wave activity is performed for Fermi 2 (Reference 20). The following discussion in Section 3.4 is based on information in Reference 20.

The wave runup analysis considers waves approaching from the northeast and the east and also considered site specific aspects of the plant layout including the presence of other buildings and the security barrier to the north of the plant. The highest runup elevation was calculated to be 591.5. Runup elevations cited in the USFAR for the Fermi 2 plant (Reference 2) range from 593 to 598 ft. Therefore, the analysis results are less than the wave runup elevations in the current licensing basis.

The methodology used in the wave runup analysis is similar to the methods used in Reference 3, Section 2.4.5.3, and is consistent with the recent NRC guidance in Reference 15.

Wave runup is evaluated to determine the wind-induced wave runup under Probable Maximum Wind Storm (PMWS) winds. Wave runup is calculated using the Automated Coastal Engineering System (ACES) model. Wave transmission and wave runup modules in the ACES model were derived from physical model studies originally conducted for specific structures and wave climates.

Two profiles were developed to describe the nearshore and shallow onshore areas. For purposes of the wave transmission and wave runup analysis the following areas were defined. Slopes are reported as Horizontal: Vertical (H: V).

- Nearshore – the area from 3.3 feet depth Mean Low Water (MLW) to 0 feet depth MLW (567.4 to 570.7). This area is between the point used to describe the waves at the shore (from STWAVE model) to the base of the seawall.
- Seawall – the area of onshore protection from an elevation of 571 feet to 583 feet, with a slope of 3H: 1V to 2H: 1V (Reference 3).
- Onshore - the area immediately behind the seawall. This area is approximately flat; elevations in this area vary from about 582 to 583 (Reference 3).

In addition, the following features were considered:

- Security Barrier – The security barrier to the north of the plant could affect wave transmission for those waves approaching from any directions that are north of east.
- Proximity of Other Structures – The RHR complex and Reactor Building would be protected from waves approaching directly from the east. However, they would be subjected to diffracted waves from the east.

The following assumptions were used for determining the wave runup. These assumptions are inherent in the ACES model and provide a conservative determination of wave runup and over-topping.

e. Waves are normally incident to the structure, and unbroken in the vicinity of the structure toe.
f. Waves are specified at the structure location.
g. All structure types are considered to be impermeable.
h. For sloped structures the crest of the structure must be above the still-water level.
i. For vertical and composite structures, partial and complete submersion for the structure is considered.
j. Runup estimates on sloped structures require the assumption of infinite structure height and a simple plane slope.

The following approach is used to calculate wave runup.

1. Results from the STWAVE model

As part of the probable maximum wind-driven surge flood analysis in Reference 3, Section 2.4.5, wave calculations were made using STWAVE. Several points that were closest to shore were examined to determine the highest waves generated. The modeling for Reference 3 provides results that can be used for the Fermi 2 analysis, showing that the highest waves generated (H_{mo}) from the northeast have a height of 12.3 feet with a peak spectral period (Tp) of 11.1 sec. For waves from the east, the wave height (H_{mo}) is 11.58 feet with a wave period (Tp) of 11.1 sec.

2. Simulate wave transmission across the nearshore and onshore areas.

The STWAVE model is not suitable for calculation of wave transmission across steep slopes; normal beach processes slopes greater than 10 H: 1V are considered steep. The seawall portion of the shore has slopes of 2 H: 1V to 3 H: 1V. Wave transmission across the nearshore and onshore areas considered the effects of changes in wave length, breaking waves due to shallow depths; the security barrier which would act as a submerged breakwater; and wave diffraction.

As waves move onshore the wavelength decreases so first the appropriate wave length must be calculated. According to Equation II-1-10 (Reference 16) the wave length is calculated as:

$L = \{ g/2\pi * T^2 \tanh (2\pi d/L) \}$.

Where: L = wave length
g = acceleration of gravity
T = wave period
d = local water depth

It is likely that the wave period would be reduced during wave breaking and transformation, however, according to Reference 16 there are no widely accepted theoretical methods for determining changes in wave period. Therefore, for the Fermi 2 analysis the wave period was assumed to remain unchanged; which is conservative.

Wavelengths associated with various points in the lake are shown in Table 3.4-1 below.

Table 3.4-1
Wave Lengths

Location	Depth (ft)	Wave Length (ft)
Deepwater	N/A	631
STWAVE point	19.2	267
Seawall	15.9	245
Barrier	4.6	134
Buildings	3.6	119

3. Calculate Breaking Wave Characteristics.

Maximum wave heights are constrained by the relative depth (ratio of wave height to water depth) and by wave steepness (ratio of wave height to wave length). Breaking wave heights were calculated according to procedures in Reference 16. Specifically equation II-4-11 was used to calculate the zero-moment wave height ($H_{mo,b}$) at the time of breaking, using the modified Miche criterion which is the same equation used by the STWAVE model. This equation represented both depth and steepness-induced wave breaking. Although not exactly equivalent in definition, the zero-moment wave height is generally considered to be equivalent to the significant wave height.

The equation used is:

$H_{mo,b} = 0.1 \text{ L} \tanh(kd)$

Where: k = wave number defined as $2\pi/L$
d = water depth

Breaking wave heights at the toe of the seawall, at the security barrier and the buildings are shown in Table 3.4-2 below.

Table 3.4-2
Breaking Wave Heights

Location	Depth (ft)	Wave Height (ft)
Seawall	15.9	9.49
Barrier	4.6	2.84
Buildings	3.6	2.24

4. Calculation of Wave Runup

The wave characteristics calculated at the RHR complex and Reactor Building from the STWAVE model are used as inputs to the ACES model to calculate wave runup. The calculations were first made without consideration of wave diffraction due to the presence of other buildings or the effect that the security barrier would have on waves. The effects of diffraction were then analyzed using the wave transformation module of ACES. The new wave characteristics were then used as inputs to the wave runup module of ACES for both the RHR complex and the Reactor Building.

The security barrier will act as a submerged breakwater and its effect on the waves was calculated by applying a reduction factor based on Reference 16. Equation VI-5-54 from Reference 16 defines the transmission coefficient (Ct) for irregular head-on waves for submerged breakwaters.

$$Ct = \{ 0.031 (Hs / D_{50}) - 0.24 \} (Rc / D_{50}) + b$$

Where: Ct = relative wave heights of the transmitted wave to the incident wave (Ht/Hs)
Hs = significant wave height of incident waves; considered equivalent to H_{mo}
D_{50} = median of nominal diameter of rocks for design conditions
Rc = freeboard, negative for submerged breakwaters
$b = -5.42 * Sop + 0.0323 (Hs/D_{50}) - 0.0017 (B/D_{50})^{1.84} + 0.51$
B = width of the crest
Sop = deepwater wave steepness corresponding to peak period.

The wave characteristics calculated for the buildings were used as inputs to the ACES model to calculate wave runup for the case of no diffraction and no effect from the security barrier. Inputs and calculated outputs for this condition are shown on Figure 6.

Waves approaching the site from the east will be diffracted around the buildings to the east of the RHR complex and the Reactor Building. To simulate this effect the wave transformation module of ACES was used. Several configurations can be used in the model including a wedge of varying angles. The buildings were simulated as a wedge with an angle of 70 degrees. The site layout is shown on Figure 7. The configuration used by the model is shown on Figure 8. Results of the diffraction and runup calculations for waves approaching from the east are summarized in Table 3.4-3, below:

Table 3.4-3
Wave Runup Elevations for Wave Diffraction

Parameter	RHR Complex	Reactor Building
Incident Wave Height (ft)	2.24	2.24
Incident Wave Period (sec)	11.1	11.1
Modified Wave Height (ft)	1.0	0.9
Runup from Modified Wave (ft)	2.0	1.9
Elevation of Runup (ft)	588.6	588.5

Waves approaching the site from any direction north of east will cross over the security barriers before reaching the buildings. The elevation of the top of the security barrier is approximately 585.4. Therefore, during the postulated maximum stillwater elevation there would be about 1.2 feet of water above the security barrier which would then act as a submerged breakwater.

The ACES model cannot simulate submerged breakwaters so equations from Reference 16 were used in the analysis. The ACES model was used to simulate a case assuming that the barrier was exactly at the water surface. For this case the transmitted wave height would be 1.1 ft. This represents a transmission coefficient (Ct) of 0.39 (Ht/Hs = 1.1/ 2.84 = 0.39). The Ct for a submerged breakwater would be greater.

Several different equations are presented in the Reference 16 to calculate wave transmission coefficients based on the breakwater characteristics. The equation for a submerged breakwater is for a conventional rock revetment type of structure and the median rock size is a required input to the equation. This does not exactly match the blocks used for the barrier. To address the difference, the size of the rock was changed to provide a conservative assumption. Larger rock sizes which are closer to the actual size of the block used in the barrier produced smaller Ct values and therefore smaller transmitted waves. A diameter of 1 foot was used in the analysis.

The calculated wave transmission and wave runup are the same for both buildings. Results of the analysis conducted to determine the effects of the barrier are summarized in Table 3.4-4, below:

Table 3.4-4
Wave Runup Elevations Accounting for Security Barrier

Parameter	Both Structures
Incident Wave Height (ft), approaching barrier	2.84
Incident Wave Period (sec)	11.1
Transmission Coefficient (Ct)	0.75
Transmitted Wave Height (ft)	2.13
Wave Runup (ft)	5.1
Elevation of Runup (ft)	591.7

In addition to evaluating wave runup, hydrodynamic forces, effects of erosion, and debris impacts due to wave activity are also considered. These are discussed below.

Hydrodynamic Forces

As described in Reference 2, Section 2.4.5.6.3, the critical static pressure and thrust occur under the broken wave conditions, whereas the critical dynamic pressure and thrust occur under the breaking wave conditions for an assumed slope of 20:1 and the minimum wave periods of 3.4 to 4.5 sec. As shown in Reference 2, Figure 2.4-18 (Wave Pressure and Forces Against Reactor Building) and Figure 2.4-19 (Wave Pressure and Forces Against RHR Complex), the pressure and force analyses are based on both 3 ft and 5.4 ft wave heights; which correspond to wave runup elevations of 593 ft and 598 ft (per Reference 2, Section 2.4.5.6.2). As shown above the maximum calculated transmitted wave height is 2.13 feet at the buildings and the maximum wave runup elevation 591.7 ft. These calculated values are well less than the corresponding values used in Reference 2 to determine the pressures and forces. Therefore, the calculated pressures and forces used in Reference 2 for structural design in the current licensing basis are limiting.

<u>Effects of Erosion of the Shore Protection Barrier</u>
The equation used to calculate the stability of a submerged breakwater is:

$$(h'c / h) = (2.1 + 0.1\ S)\ \exp(-0.14\ N^*s)$$

Where: h'c = height of the structure over the seabed
h = water depth
S = safety factor
$N^*s = (Hs / \Delta\ D_{50})\ sp^{-1/3}$
Hs = significant wave height
sp = wave steepness (Hs/L)
L = wave length
$\Delta = (\rho s / \rho w) - 1$
ρs = density of rock
ρw = density of water

N*s and D_{50} values were calculated for S values of 2, 5, and 8 corresponding to initial damage and intermediate damage. The results are shown in Table 3.4-5, below.

Table 3.4-5

S	N*s	D_{50} (ft)
2	7.783	2.1
5	8.659	1.8
8	9.439	1.8

The diameters of the rocks on the seawall are in the range of 3 to 4 feet. Therefore, the shore protection barrier would be protected from erosion during the postulated flood.

Additional sensitivity analyses were performed using the ACES model and the wave characteristics defined at the toe of the slope. For this case, it was assumed that the seawall extended to an elevation above the postulated flood elevation. The input values included water depth of 15.9 ft, wave height of 9.49 ft, and wave period of 11.1 sec. The model determines D_{50} values for safety factors of 2, 5, and 8. For the safety factor of 2 the required D_{50} would be 4.1 ft. D_{50} values of 3.2 to 3.4 ft would result in intermediate damage, without failure of the seawall. Therefore, in this case, some damage to the seawall could occur but the seawall would not fail. Some damage to the seawall during a major storm is consistent with the design basis for the Fermi 2 plant.

This is a conservative analysis because the seawall would actually be submerged during the maximum flood event. The model assumes that waves would be breaking on the face of the revetment. The point where waves break on the revetment, the stillwater level, is the area most subject to damage. For the postulated flood condition, the entire seawall would be underwater and therefore, not subject to the full force of the breaking waves associated with this water depth. During a lesser surge event, the seawall might not be submerged but for that condition, the wave height would be less than that used in this analysis (9.49 ft).

Another condition was examined for potential stability of the seawall when the stillwater level is just at the top of the seawall. This is not the maximum flood level but a condition that could occur during the flooding event. In this case the water depth at the toe of the seawall would be 12.3 ft. Using the equation for breaking wave heights, the wave height would be 7.44 ft and the wave period is 11.1. For a safety factor of 2, the required D_{50} value is 3.3 ft. For an intermediate level of damage, the required D_{50} values range from 2.5 to 2.8 ft. These diameters are less than the diameter of rock on the seawall.

The extent of predicted damage to the seawall is consistent with that described in Reference 2, Section 2.4.5.7; which states that the original design of the shore barrier allows for some shore barrier damage (i.e., some rock displacement during major storms). The Technical Requirements Manual requires that the shore barrier be inspected annually and after major storms. In addition, Reference 2 states that damage to the shore barrier will not enable waves larger than 5.4 ft to break against Category I structures because of their distance inland.

Impacts from Debris and Water Borne Projectiles

Analysis was performed to determine the impact forces from debris striking the structure (Reactor/Auxiliary Building and RHR Complex) walls at the breaking wave velocity (Reference 21). The analysis was performed consistent with the guidance in Reference 15. The analysis considers the effect to the SSCs important to safety from debris and water-borne projectiles during the postulated wind-driven surge event. Methodology in ASCE 7-10 (Reference 24) was used to determine the loads from debris or water borne projectiles based on a 2000 pound projectile assumed to impact the structure at the wave velocity. The resultant loads were compared to the loads for which the structures are designed (tornado missiles) to ensure that the design can accommodate water borne projectiles. The analysis results demonstrate that loads from the waterborne debris plus hydrodynamic loads are less than the loads that the structures are designed for; therefore, the results demonstrate that the structures are capable of handling the postulated loads.

3.5 Seiche

Regarding seiches, Reference 3, Section 2.4.5.2.2.3 states:

> "Seiches are standing waves of relatively long periods that occur in lakes and other water bodies. Lake Erie is subject to occasional seiches of irregular amount and duration, which sometimes result from a sudden change, or a series of intermittent periodic changes, in atmospheric pressure or wind velocity. The maximum deviations from mean lake levels at Toledo were reported in the U.S. Army Corps of Engineers Shore Protection Manual (Reference 2.4-249). The maximum recorded rise was 1.9 m (6.3 ft) and the maximum recorded fall was 2.7 m (8.9 ft) for the period from 1941 to 1981. The value of the rise is significantly less than the storm surge calculated using the Bretschneider methods ... ."

Regarding seiches at the Fermi site, NRC Safety Evaluation Report for Chapter 2 (Reference 4), Section 2.4.5.4 (top of Page 2-147), states

> "Staff reviewed the historical data seiche in Lake Erie and confirmed its effect is less than impact of surge under PMWS [Probable Maximum Wind Storm] in the site area. The staff concluded that the information was accurate and applicable to the site."

As both Fermi 3 and Fermi 2 are on the same site and share the same major hydrologic features (i.e., Lake Erie and the Swan Creek Drainage Basin), this same rationale is applicable to Fermi 2. Therefore, for Fermi 2, the maximum expected rise from a seiche is less than the storm surge described in Section 3.4, above.

Seiche events can also result in minimum lake levels at the site. As described in Reference 2, Subsection 9.2.5, the cooling water supply for safety-related systems is provided by the RHR service water system, which contains its own water reservoir and is independent of ground- or lake-water supplies. That is, the ultimate heat sink for Fermi 2 is independent of local water-level conditions.

3.6 Tsunami

Regarding tsunamis, Reference 3, Section 2.4.6 states:

> "The National Oceanographic and Atmospheric Administration (NOAA) National Geophysical Data Center (NGDC) maintains a historical tsunami database which catalogs tsunami events. The data in the NGDC database was filtered to exclude invalid events. Based on this filtering, no tsunami events were identified in the Great Lakes.
>
> "Furthermore, valid tsunami events in Lake Erie are considered unlikely based on the absence of historical large earthquakes in the Lake Erie region, a low probability of vertical displacement of the lake bed during a seismic event, the relatively shallow depth of Lake Erie, and the very gentle bottom profile of Lake Erie (particularly in the western and central basins). Observed similar phenomena have been low-amplitude seiches resulting from sudden barometric pressure differences.
>
> "Therefore, there are no potential tsunamis which could affect safety-related structures or components at Fermi 3."

In addition to the discussion in the Reference 3, Section 2.4.6, further details are provided in the response to NRC RAI 02.04.06-1 (Reference 25).

Regarding flooding due to tsunamis, the NRC Safety Evaluation Report for Chapter 2 (Reference 4), Section 2.4.6.4, states:

> "To verify applicant's conclusion, the NRC staff searched tsunami database (National Geophysical Data Center, NOAA) and found two historical events: one in the northern end of Lake Erie and the other near the Detroit River. The staff requested that the applicant conduct a thorough search for historical tsunamis in the area providing an evaluation to support the applicant's conclusion. RAI 2.4.6-1, as issued to the applicant, requested this information.
>
> "In response to RAI 2.4.6-1 (ML100330612, dated January 29, 2010), the applicant provided additional information regarding historic records in the area, indicating that the recorded

historical events were only minor disturbances or seiches and no actual tsunamis are evident. The applicant's review of historic data is complete and accurate, and the response is deemed acceptable because it meets the requirements of 10 CFR Part 100, 10 CFR 100.23(d), and 10 CFR 52.79(a)(1)(iii)."

As both Fermi 3 and Fermi 2 are on the same site and share the same major hydrologic features (i.e., Lake Erie and the Swan Creek Drainage Basin), this same rationale is applicable to Fermi 2. Therefore, there are no potential tsunamis that could affect Fermi 2 safety-related structures or components.

3.7 Ice Induced Flooding

Regarding ice induced flooding, the Reference 3, Section 2.4.7, states:

"The emergency cooling system for Fermi 3 is provided by the Ultimate Heat Sink (UHS) which does not rely on water sources external to the plant and is not affected by ice conditions. This is further described in Subsection 9.2.5. Therefore, there are no safety-related systems, structures, or components impacted by ice formations.

"Based on information in Reference 2.4-296, there is no evidence to indicate ice/snow blockage along the surrounding areas of Fermi 3. From Reference 2.4-296, the closest documented ice jams are along the Raisin River, which is approximately 10 miles south and does not intersect waterways affecting the Fermi 3 site. Therefore, flooding due to ice jam blockage is not anticipated to impact Fermi 3."

In addition to the discussion in Reference 3, Section 2.4.7, further detail (including screen shots from the Corp of Engineers Ice Jam Data Base (https://rgis.crrel.usace.army.mil/apex/f?p=273:3:1818914186479561) (Reference 22) is provided in response to NRC RAI 02.04.03-1 (Reference 26).

Regarding ice effects, the NRC Safety Evaluation Report for the Fermi 3 FSAR, Chapter 2 (Reference 4), Section 2.4.7.4, states:

"No discussion was presented on ice effects in the FSAR. The staff submitted RAI 2.4.3-1 asking for information to support the conclusion that there would be no impacts to Fermi 3 safety-related features due to ice effects. In the response to RAI 2.4.3-1 (ML092790561, dated September 30, 2009), the applicant cited checking the USACE ice jam database for historical occurrences of ice jams on Swan Creek. The applicant found no historic ice jams on Swan Creek in the ice jam database. Also, in the response to RAI 2.4.9-1, the applicant stated that no ice jams were observed on Swan Creek over the period from 1957 to the present, during which time the applicant managed the Fermi site.

"To verify the applicant's response, the staff performed a search of the USACE ice jam database and found no evidence of an historical ice jam on Swan Creek (USACE, 2010). However, in the description of the ice jam database, the USACE stated that the historical records of ice jams are primarily limited to waterways that have USGS gauging stations (USACE, 2010). There have never been continuously recording USGS gauging stations on Swan Creek, so the likelihood of an historical ice jam being recorded on Swan Creek is low. However, the applicant stated that there have been no ice jams on Swan Creek since 1957. The gauging station on the River Raisin to the south has recorded several ice jams since that time, and records of this flooding are found both on the ice jam database and in local media sources. No personal accounts or media accounts of flooding in Swan Creek due to ice jams were found. Therefore, the staff finds that the applicant's answer is acceptable in that ice jams are not likely to contribute to flooding in Swan Creek."

As both Fermi 3 and Fermi 2 are on the same site and share the same major hydrologic features (i.e., Lake Erie and the Swan Creek Drainage Basin), this same rationale is applicable to Fermi 2. Furthermore, as described in Reference 2, Subsection 9.2.5, the cooling water supply for safety-related systems is provided by the RHR service water system, which contains its own water reservoir and is independent of ground- or lake-water supplies. That is, the ultimate heat sink for Fermi 2 would not be affected by ice induced flowing on Swan Creek or Lake Erie.

3.8 Channel Migration or Diversion

Regarding channel migration or diversion, Reference 3, Section 2.4.9, states:

"Fermi 3 site and facilities are discussed in Subsection 2.4.1. No safety-related systems, structures, or components are impacted. The water supply for Fermi 3 is not obtained from channels; therefore, this subsection is not applicable from a water supply perspective.

"The topography surrounding Swan Creek slopes gently toward the creek, with the creek sloped to drain in a well established channel into Lake Erie. As identified in FSAR Subsection 2.5.1 and Subsection 2.5.4, the natural geology consists of lacustrine deposits overlying stronger glacial till (see FSAR Subsection 2.5.4, Table 2.5.4-202), which overlies bedrock. The increase of strength with depth of the subsurface soil and bedrock minimizes the possibility of a block-type failure of a stronger layer on a shallow failure plane in a weaker underlying layer; therefore, shallow large area landslides into Swan Creek are not envisioned as a plausible scenario. The shallow slope of the land surface toward the creek combined with the lack of a weak soil layer at depth also reduces the potential for the occurrence of a large area landslide into the creek.

"Along the incised banks of Swan Creek, local slope failures of the creek bank are part of the natural creek erosion process, however the size of these slope failures would be limited to near bank events which would not result in diversion of Swan Creek.

"Refer to Subsection 2.4.7 for discussion on ice jams. Ice jams are not expected to cause a diversion of Swan Creek.

"Detroit Edison has managed the Fermi site from 1957 to the present and has no record or knowledge of any human induced or natural diversions of Swan Creek occurring during that time or prior to 1957. The lack of past events suggests that it is unlikely Swan Creek would be diverted by a future event.

> "As the area surrounding Swan Creek is relatively well established, it is anticipated that future human-induced diversions of Swan Creek that would impact the Fermi 3 site are unlikely.
>
> "The Fermi 3 site is not expected to be impacted by channel diversions."

Regarding channel migration or diversion, the NRC Safety Evaluation Report for the Fermi 3 FSAR Chapter 2 (Reference 4), Section 2.4.9.3, states:

> "In the FSAR, the applicant stated that this section is not applicable to Fermi 3, as Fermi 3 does not rely on channels for water supply. The staff submitted RAI 2.4.9-1 requesting information supporting the conclusion that a diversion along Swan Creek from an ice jam, a landslide, or another mechanism is unlikely. In the response to RAI 2.4.9-1 (ML092790561, dated September 30, 2009), the applicant provided a discussion supporting the conclusion that a diversion along Swan Creek is unlikely. The applicant stated that the geology and topography of the Swan Creek watershed are not conducive to large scale landslides that could cause a channel diversion. First, the applicant described the geology as being a sequence of bedrock overlain by glacial till deposits overlain by lacustrine deposits. Then the applicant stated that the deposits increase in strength with depth and that the topography of the watershed is not steep, making the chances of a large area landslide caused by a failing lower layer small. The applicant stated that the banks of Swan Creek do experience small failures, but they would not be of large enough size to divert Swan Creek. Then the applicant referred to FSAR Section 2.4.7 and the response to RAI 2.4.3-1 (ML092790561, dated September 30, 2009) to support the conclusion that it is unlikely that an ice jam would occur on Swan Creek and cause a diversion. The applicant also stated that no manmade or natural diversions were observed over the period from 1957 to the present, during which time the applicant managed the Fermi site. The staff found the applicant's response acceptable."

As both Fermi 3 and Fermi 2 are on the same site and share the same major hydrologic features (i.e., Lake Erie and the Swan Creek Drainage Basin), this same rationale is applicable to Fermi 2. Therefore, there are no potential channel migration or diversion scenarios that could affect Fermi 2 safety-related structures or components.

3.9 Combined Effect Flood

Flood analyses accounting for combined effects are performed consistent with Reference 5. Details regarding the analyses of combined effects are provided in the discussion of the PMF and Wind Driven Surge Flood analyses for Fermi 3 in Sections 3.2 and 3.4, above.

4. COMPARISON WITH CURRENT DESIGN BASIS

The attached Table 4.0-1 provides a comparison of current and reevaluated flood causing mechanisms for Fermi 2 and includes a comparison of the limiting results to the design basis flood protection. As shown in Table 4.0-1, the results from the updated flood evaluations analyses for Fermi 2 are less than the design bases flood protection. Therefore, an interim evaluation is not required and there are no additional actions taken or planned. In addition, based on these results and the guidance in Reference 18, Fermi 2 is not required to perform an Integrated Assessment.

As described in Reference 17, Available Physical Margins (APM) were collected and documented on the Walkdown Record Forms; which are retained and available for NRC audits and inspections. As documented in Reference 17, no conditions related to small APM with large consequences (indicative of a potential cliff-edge effect) were identified. The approach followed for determining APM was in accordance with NEI 12-07 (Reference 27).

The new flood hazard analyses, described in this FHRR, did not provide results that degraded the identified APM.

5. REFERENCES

1. NRC Letter to All Power Reactor Licensees, "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near Term Task Force Review of Insights from the Fukushima Dia-Ichi Accident [ML12053A340].

2. Fermi 2 Updated Final Safety Analysis Report (UFSAR), Revision 18.

3. Fermi 3 Final Safety Analysis Report (FSAR), Section 2.4, "Hydrology," Rev. 4 [ML12095A082].

4. NRC Safety Evaluation Report (with no open items) for FSAR Chapter 2, "Site Characteristics," dated 7/20/2012 [ML121020116].

5. American Nuclear Society, "Determining Design Basis Flooding at Power Reactor Sites," an American National Standard, ANSI/ANS-2.8-1992.

6. NUREG/CR-7046 PNNL-20091, U.S. NRC, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, Published November 2011.

7. NOAA, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian" Hydrometeorological Report No. 51, June 1978.

8. NOAA, "Application of Probable Maximum Precipitation Estimates-United States East of the 105th Meridian "Hydrometeorological Report No. 52, August 1982.

9. U.S. Army Corps of Engineers, "Runoff from Snowmelt, EM 1110-2-1406", March 1998.

10. US Department of Commerce, Weather Bureau, "TP 40 Rainfall Frequency Atlas of the United States for Duration from 30 minutes to 24 hours and Return Periods for 1 to 100 years", dated May 1961.

11. USDA, Natural Resources Conservation Service, TR-55, "Urban Hydrology for Small Watersheds," dated June 1986.

12. Daugherty, R.L., Franzini, J.B. and Finnemore, E.J., "Fluid Mechanics Engineering Applications," Eighth Edition, MacGraw Hill, 1985.

13. Kansas City Metropolitan Chapter, "American Public Works Association Standard Specifications & Design Criteria," Section 5600 Storm Drainage Systems & Facilities, February 16, 2011.

14. Ven Te Chow, "Open Channel Hydraulics," 1959.

15. U.S NRC, Japan Lessons-Learned Project Directorate, JLD-ISG-2012-06, "Guidance for Performing a Tsunami , Surge, or Seiche Hazard Assessment," Interim Staff Guidance. January 4, 2013.

16. U.S. Army Corps of Engineers. "Coastal Engineering Manual," Engineer Manual 1110-2-1100, 2002.

17. Letter No. NRC-12-0076, Detroit Edison Letter to the NRC dated November 26, 2012, "Detroit Edison Response to March 12, 2012 Information Requested Regarding Flood Protection Walkdowns."

18. NRC Letter to Mr. Joseph E. Pollock, Executive Director, Strategic Programs, Nuclear Energy Institute, Subject: Trigger Conditions for Performing an Integrated Assessment and Due Date for Response," dated December 3, 2012.

19. Calculation 177910.51.1001, "Local Intense PMP Hydrology and Hydraulics," Revision 0.

20. Calculation 177910.51.1002, "Fermi 2 Maximum Wave Run-Up Elevations," Revision 0.

21. Calculation 177910.51.1003, "Wave Debris Load on Vertical Wall," Revision 1.

22. Corps of Engineers Ice Jam Database https://rgis.crrel.usace.army.mil/apex/f?p=273:3:1818914186479561, accessed September 2, 2009.

23. CorpsMap, National Inventory of Dams, https://nid.usace.army.mil, accessed September 2, 2009.

24. Minimum Design Loadings for Buildings and Other Structures, ASCE 7-10, American Society of Civil Engineers, 2010.

25. Detroit Edison Letter NRC3-10-0007, dated January 29, 2010, "Detroit Edison Company Response to NRC Request for Additional Information Letter No. 19." [ML100330612]

26. Detroit Edison letter NRC3-09-0027, dated September 30, 2009, "Detroit Edison Company Response to NRC Request for Additional Information Letter No. 10." [ML092790561]

27. NEI 12-07, Revision 0-A, "Guidelines for Performing Verification Walkdowns of Plant Flood Protection Features," dated May 2012. [Accession No. ML12173A215]

Table 4.0-1
Current and Reevaluated Flood Evaluation Results vs. Flood Protection

Flood Causing Mechanism	Current	Reevaluated	Flood Protection
Local Intense Precipitation (PMP)	The estimated PMP would not result in adverse plant site flooding. The temporary local water buildup due to the failure of the plant drainage system will be substantially lower than the flood elevation attributed to the maximum storm surge.	As described in Section 3.1, above, an updated PMP analyses was performed for Fermi 2. The results show a maximum predicted water surface elevation of 583.4 ft.	The predicted water surface elevation is less than the height of the door sills for the Class I structures (583.5 ft).
Flooding in Streams and Rivers (PMF)	As described in Fermi 2 UFSAR Section 2.4.3.5, predicted water elevation is 577.3 ft for a peak flood flow of 89,000 cfs, and an elevation of 579.1 ft is predicted for a higher flow rate of 115,000 cfs.	As described in Section 3.2, above, a PMF performed for the Fermi 3 COLA is applicable to Fermi 2. This updated PMF analysis shows that the maximum flood level is 579.5 ft and is approximately 3.5 ft below site grade (combined effect of PMF plus 25-year surge). The maximum combined effect flood level is 586.6 ft; which is due to the effect from combining the probable maximum surge with the 25 year flood.	The Category I reactor/auxiliary building is designed for a stillwater elevation of 588.0 ft, and wave runup elevation 593 ft. The RHR complex is designed for a stillwater elevation of 590.0 ft, and wave runup elevation 598 ft. Flood protection is greater than the results from the Fermi 2 current licensing bases analyses and from the Fermi 3 analyses.
Dam Breaches and Failures	As described in the Fermi 2 UFSAR, Section 2.4.4, there are no dams on Swan Creek or other streams or rivers in southeastern Michigan whose failure would affect water levels in Lake Erie along the plant shoreline.	As described in Section 3.3, above, flooding due to dam breach or failure is not a concern for Fermi 2 as there are no upstream dams.	N/A

Table 4.0-1
Current and Reevaluated Flood Evaluation Results vs. Flood Protection

Flood Causing Mechanism	Current	Reevaluated	Flood Protection
Storm Surge and Wave Run-Up	As described in the Fermi 2 UFSAR Section 2.4.5.3, the maximum predicted stillwater elevation is 586.9 ft. As described in the Fermi 2 UFSAR Section 2.4.5.6.2, the maximum run-up elevations on the exposed north faces of the reactor/auxiliary building and the RHR complex are 593.0 and 598.0 ft for the 3.0-ft and 5.4-ft waves, respectively. The maximum runup elevation on the exposed south faces of the reactor/ auxiliary building and the RHR complex, the exposed east face of the RHR complex, and the west face of the reactor/auxiliary building is 593.0 ft for the 3.0-ft wave.	As described in Section 3.4, above, an analysis of flooding due to a wind-driven surge from Lake Erie performed for the Fermi 3 COLA is applicable to Fermi 2. This updated surge analysis shows that the maximum flood level is 586.6 ft. Due to differences between Fermi 3 and Fermi 2, the wave run-up analyses performed for Fermi 3 cannot be directly applied to Fermi 2. Thus, a new wave run-up analysis was performed for Fermi 2. The analysis predicts a maximum wave runup elevation of 591.7 ft.	The Category I reactor/auxiliary building is designed for a stillwater elevation of 588.0 ft, and wave runup elevation 593 ft. The RHR complex is designed for a stillwater elevation of 590.0 ft, and wave runup elevation 598 ft. Flood protection for the maximum stillwater elevation is greater than the results from the Fermi 2 current licensing bases analyses and from the Fermi 3 analyses.
Seiche	As described in the Fermi 2 UFSAR Section 2.4.5.2.3, for design purposes, no rise in water elevation from a seiche is considered.	As described in Section 3.5, above, the results from a seiche are much less than the results from the storm surge flood.	Bounded by protection provided for Storm Surge flood.
Tsunami	As described in the Fermi 2 UFSAR, Section 2.4.6, flooding due a tsunami is not considered.	As described in Section 3.6, flooding due to a tsunami is not considered at the Fermi site.	N/A
Ice Induced Flooding	As described in the Fermi 2 UFSAR, Section 2.4.7, ice-induced flooding is not considered.	As described in Section 3.7, above, flooding due to ice dams is not considered as there is no history of ice dams in Swan Creek and surrounding areas.	N/A

Table 4.0-1
Current and Reevaluated Flood Evaluation Results vs. Flood Protection

Flood Causing Mechanism	Current	Reevaluated	Flood Protection
Channel Migration and Diversion	As described in the Fermi 2 UFSAR, Section 2.4.9, flooding due to channel diversion is not considered.	As described in Section 3.8, above, flooding due to channel migration or diversion is not considered based on the geology and topography of the area.	N/A
Combined Effect Flood	Not specifically addressed in the Fermi 2 UFSAR. ANSI/ANS-2.8-1992 issued subsequent to Fermi 2 licensing.	Combined effects flooding are addressed as part of the PMF and Storm Surge flood events (Sections 3.2 and 3.4).	Refer to above discussion for PMF and Storm Surge.

Figure 1
Fermi Site Showing Proximity of Proposed Fermi 3 to Existing Fermi 2
(Extracted from Fermi 3 FSAR Figure 2.1-203)

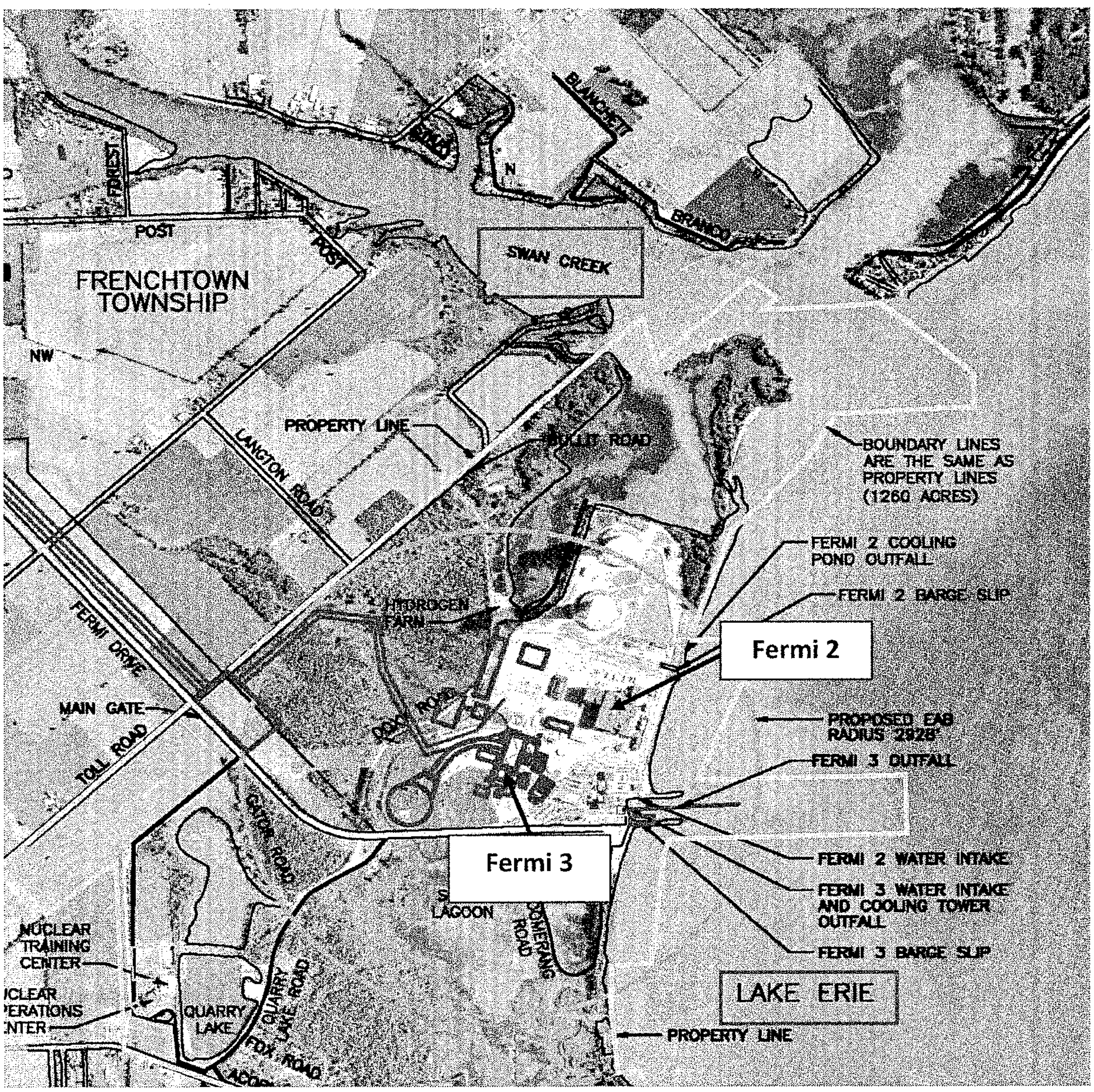

Figure 2
1.78 sq. mi. Local Watershed and 51-acre Plant Site Boundaries

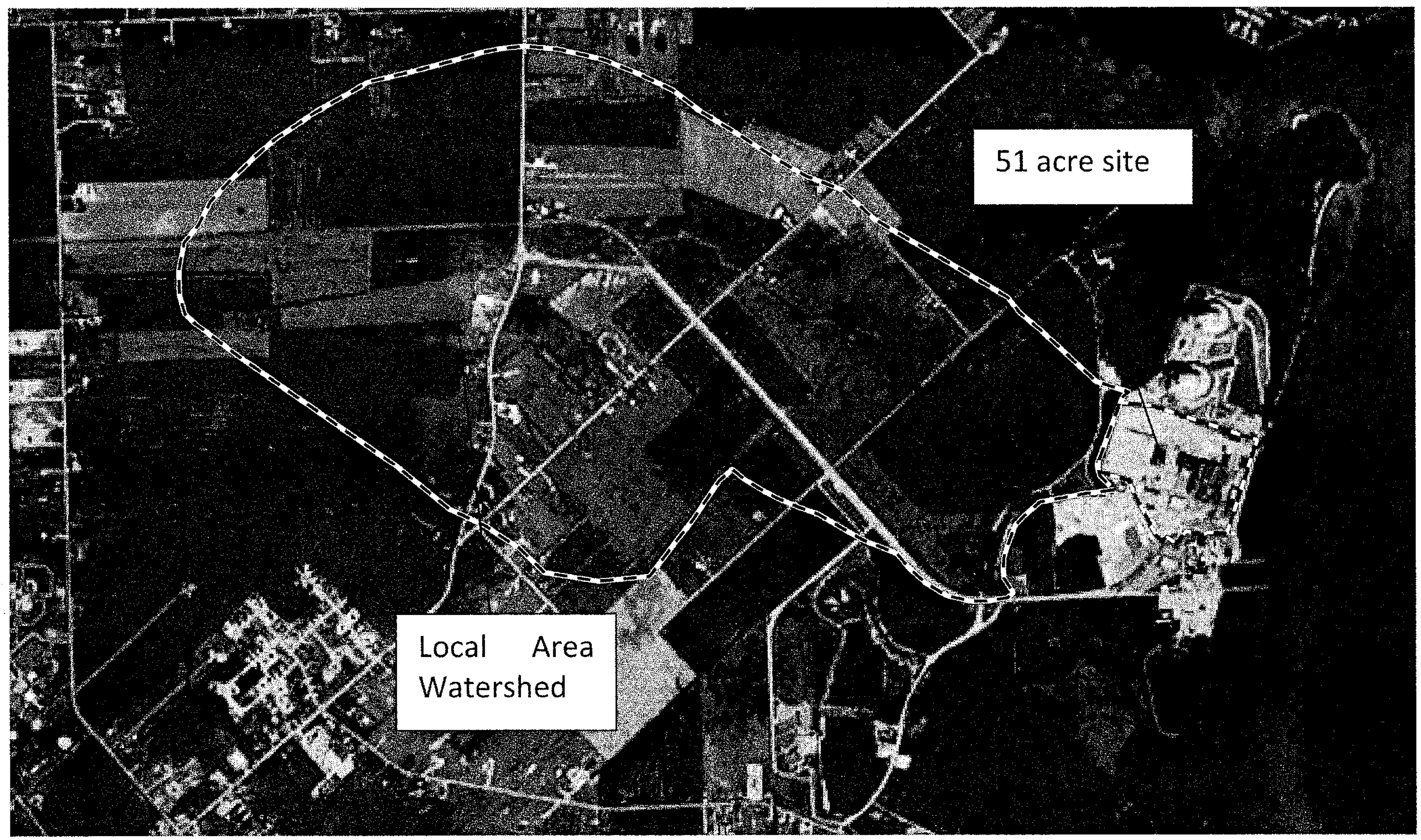

Figure 3
Fermi 2 Local Intense PMP Duration-Intensity Curve

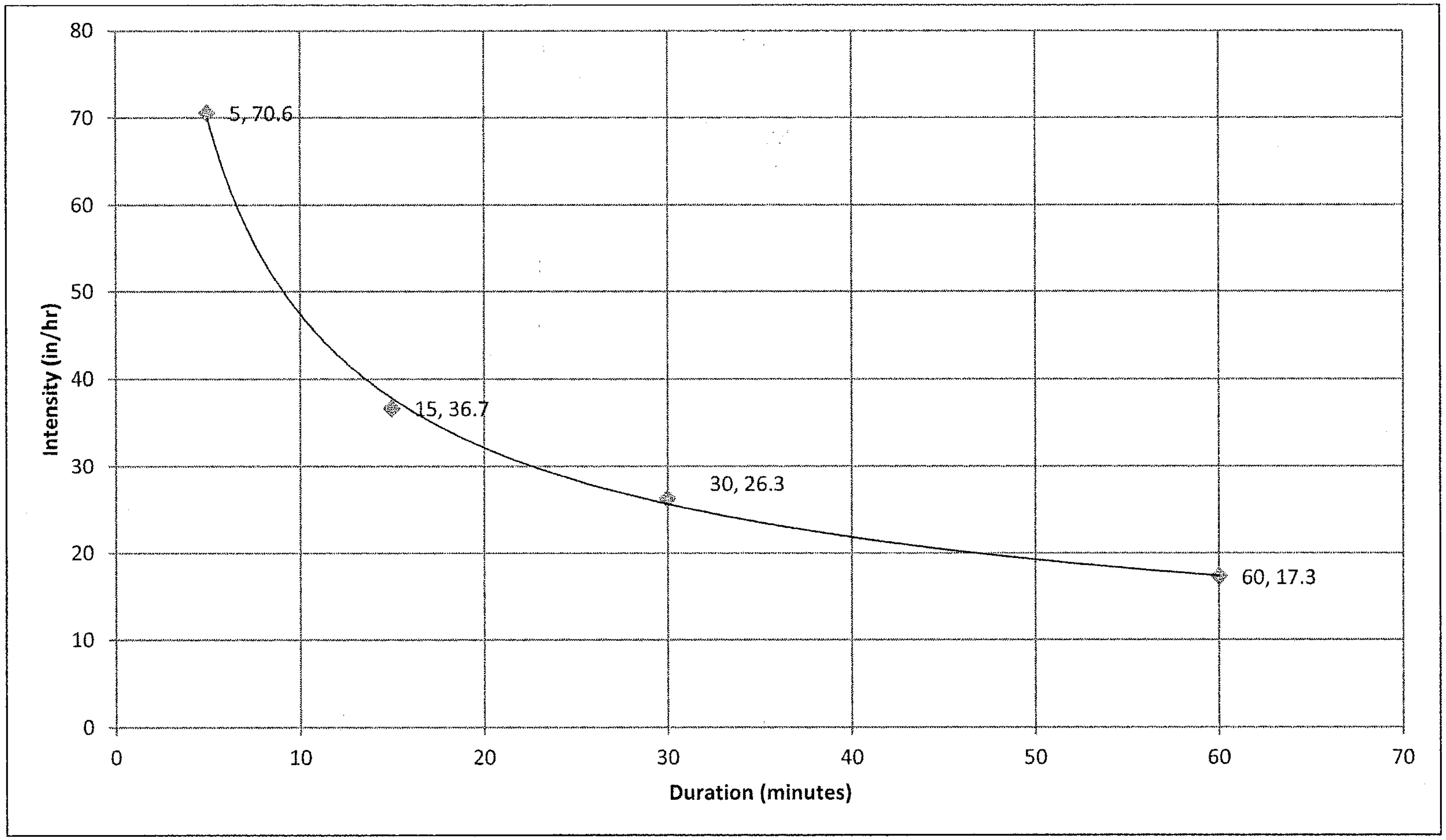

Figure 4
Site Characterization with Structures Depicted

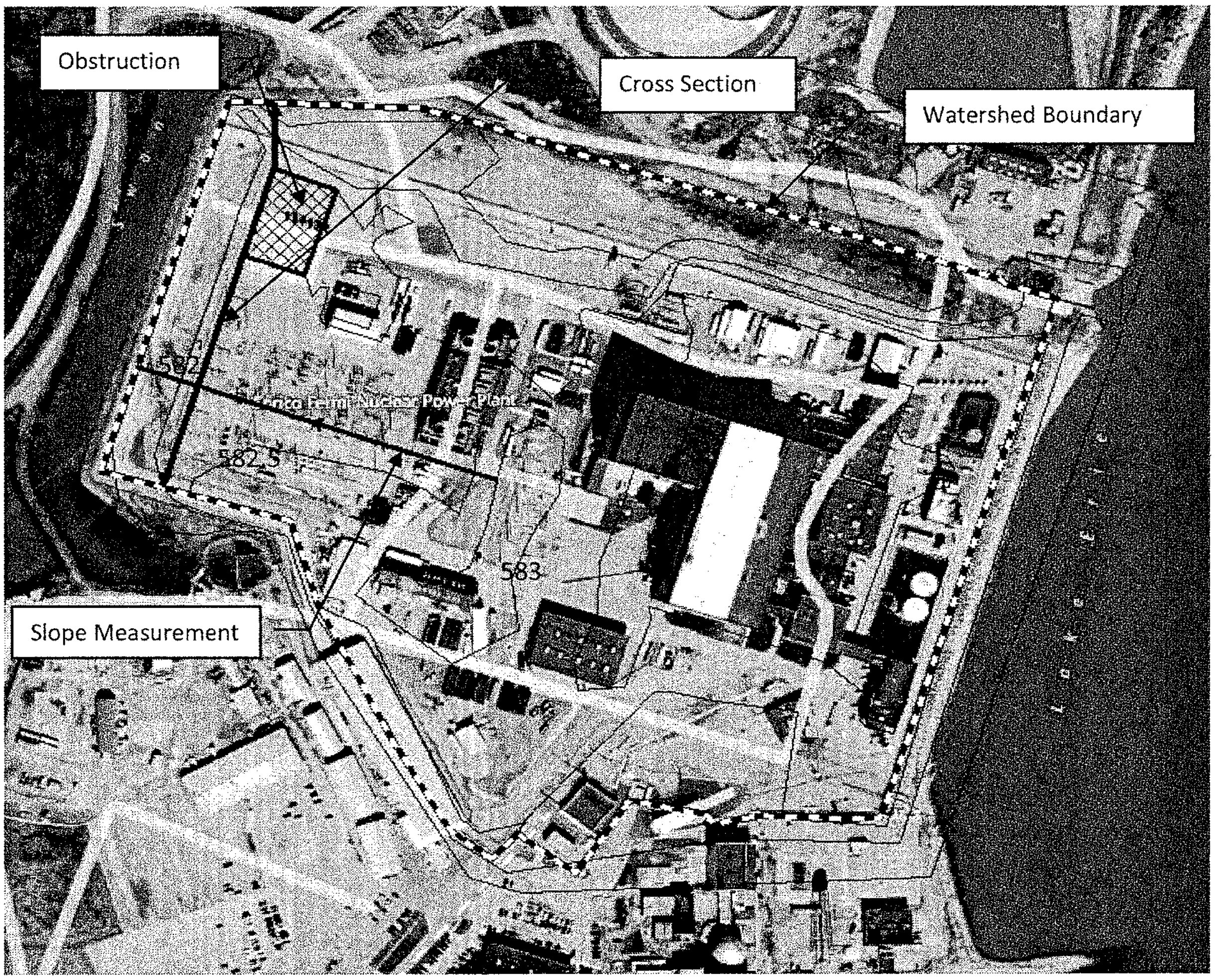

Figure 5
Site Characterization without Structures Depicted

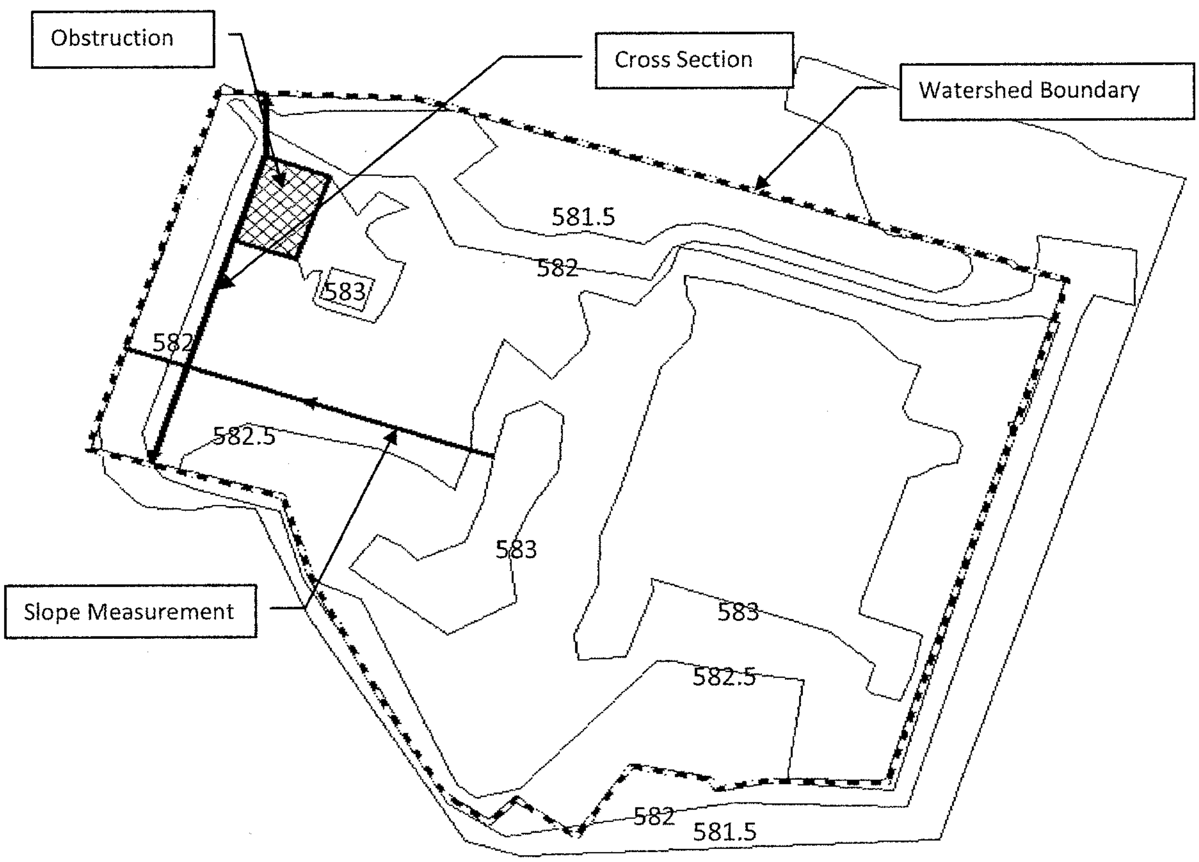

Figure 6
Inputs and Results for ACES Base Case Condition

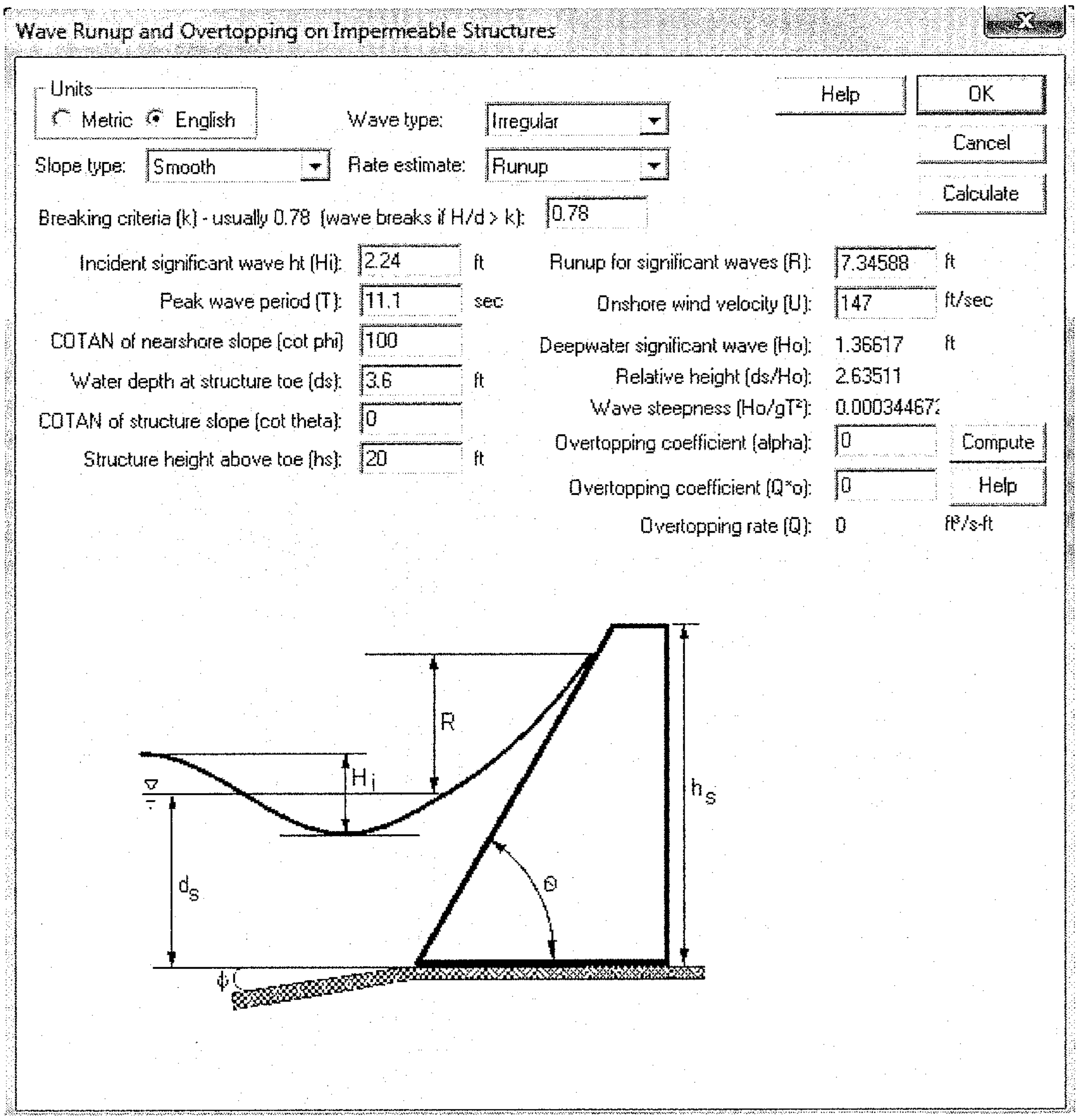

Figure 7
Configuration for Diffraction Analysis

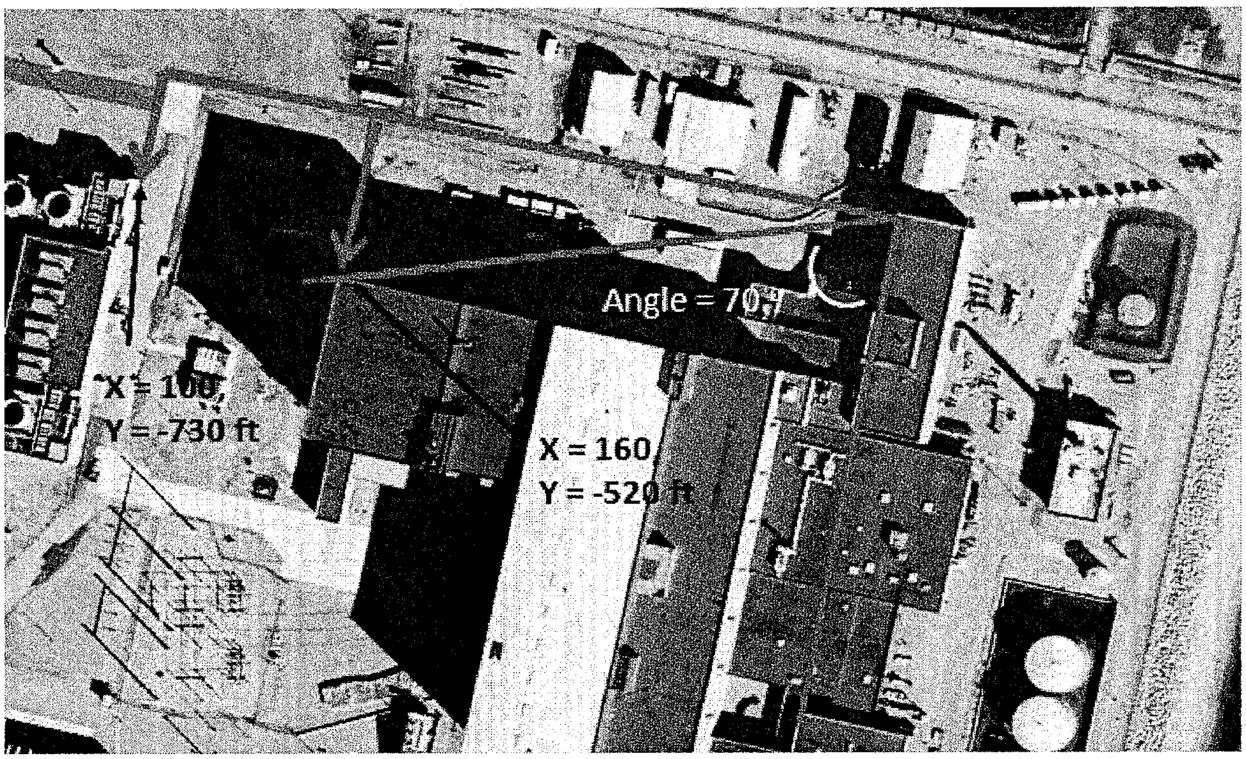

Figure 8
Modeling Used for Diffraction Analysis